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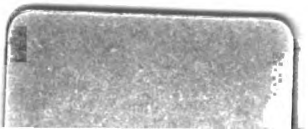

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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

August, 1902

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ERRATUM.

Vol. XLVIII, page 314, in title: For "W. C. Hammond" read "W. C. Hammatt."

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

No. 916.

MECHANICAL INSTALLATION IN THE MODERN OFFICE BUILDING.*

By Charles G. Darrach, M. Am. Soc. C. E.

With Discussion by Messrs. Robert C. Clarkson, John W. Hill, William Copeland Furber, Reginald Pelham Bolton and Charles G. Darrach.

The mechanical installation in the modern office building presents to the engineer a field for the best utilization of his energies. The office building of to-day contains all the conveniences and necessities of a great city, the microcosm of which it is. In addition to the engineering features of a city, a denser population must be provided for, and the varied interests and conveniences of each tenant must be thoughtfully studied, in order to make the little city a success.

* Presented at the meeting of October 2d, 1901.

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In the general arrangement of the apparatus, there must be considered economy of space, comfort, safety and economy in operation, convenience in distribution, adaptability to modification, ease in repairs and beauty of design; and the machinery must be constructed at the least cost, to be operated at the minimum expense.

The object of this paper is to present for discussion the various necessities in the modern office building and some general rules for practice which experience indicates.

As far as possible, a building should be an individual, depending upon no other, public or private; and it is possible to be independent of everything except primary supplies and sewage disposal.

The necessities are:

Heating and Ventilation,
Lighting,
Water Supplies and Sewage Disposal,
Policing and Fire Protection,
Transportation,
Communication with the outside world.

The source of power almost invariably adopted is steam, as it can be utilized for heating the building after doing its work in the engines. The boiler power required to heat under the most severe conditions is, except in some very high buildings, generally sufficient to operate the machinery. Practice indicates the installation of from 140 to 200 H.-P. for maximum use per 1 000 000 cu. ft. of contents, with one additional boiler unit in reserve.

Heating and Ventilation.—In this country 70° Fahr. is the standard temperature maintained in the building during the heating period.

Except in mountain regions, the lowest temperatures are found in the Mississippi water-shed. The minimum record at Winnipeg, the extreme north, was —50°, St. Paul —41°, Chicago —23°, Indianapolis —25°, St. Louis —22°, New Orleans, the extreme south + 5°, Pittsburg, at the east, —20°, and Denver, at the west, —29 degrees.

On the Atlantic Coast the minimum ranges from —20° on the Maine Coast to —6° at New York and Philadelphia, and to 0° as far south as Alabama.

Inland, the temperature falls to —30° in Central Maine and —20° in Central New York and Pennsylvania.

The minimum temperature on the Pacific Coast at the same latitude is much higher than on the Atlantic Coast, that at San

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Francisco being + 20° as compared with — 6° at New York and Philadelphia.

In New York and Philadelphia the outside temperature falls below 70° during nearly 70% of the yearly cycle, and to make a building comfortable it must be heated to 70° Fahr. from September 1st to May 15th. From December 1st to March 1st the average temperature may be as low as 30°; the temperature often falls to + 10°, and sometimes below zero. Wind storms of 25 to 45 miles per hour are not infrequent.

The heating plant should be sufficient to maintain the standard temperature of 70° in the coldest weather, and provision should be made to protect the building from the effect of wind storms.

It is more difficult to heat a building during high wind with moderately low temperature than during a calm day and minimum temperature. Leakage through window frames and sashes causes the greatest loss. During a high wind a film of warm air on the outside of the building is blown off and the radiation from the exposed surface is thereby increased. Tight window frames and good weather strips prevent most of the loss from leakage. The loss in radiation, due to wind, must be made up by either higher steam temperatures in the radiators or by additional radiating surface.

To illustrate the effect of wind storms the writer made the following observations: The rooms on the windward side of a tall building when the outside temperature was 17°, and wind at 25 miles per hour, could not be heated to over 62° with steam in the radiators at a temperature of 218°. The following day the room was heated to 71° with steam at 215°, although the temperature outside had fallen to 9°; but the wind had also fallen to 12 miles per hour. The windows were subsequently weather-stripped, and during the following winter there was no difficulty whatever in heating the rooms to above 70° with steam at 210° during wind storms of 35 miles per hour and with the temperature at 15 degrees.

A comparison of the coal burned during 60 days, when the average temperature was 30°, showed a saving of from 25 to 30% after the building had been weather-stripped.

From the writer's observations, buildings with a large amount of glass exposure can generally be heated, during 75 to 80% of the heating period, by the exhaust steam from the engines driving the ma-

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chinery, live steam being used during very cold or windy days, at night, and in the early morning before the building is occupied.

To illustrate, Table No. 1, showing the results in two buildings, may be of use.

Both buildings are heated with direct radiation above the first story; the first story and basement are heated with warm air and ventilated with exhaust fans. Building "A" is ventilated above the first floor with exhaust fans.

TABLE No. 1.

	Building "A."	Building "B."
	Coal for Heating only.	Coal for Power and Heating.
Cubic contents	1 300 000 cu. ft.	1 515 000 cu. ft.
Stories	12	15
Height	171 ft.	208 ft.
Width	58 "	71 "
Length	111 "	100 "
Glass surface	12 000 sq. ft.	28 500 sq. ft.
Wall surface	53 000 "	41 000 "
Roof surface	8 000 "	8 700 "
Coal per day	2.74 tons.	6 tons.
Coal per day, 1 000 cu. ft.	4.72 lbs.	8.87 lbs.
Coal per day, 1 000 cu. ft., equated for glass surface	4.72 "	5.88 "
Time of test	60 days.	60 days.
Average temperature for 60 days	30° Fahr.	30° Fahr.
Minimum temperature	8° "	8° "
Maximum winds	42 miles per hour.	42 miles per hour.

The windows of Building "A" are weather-stripped and in Building "B" they are without weather strips.

The amount of coal used for heating Building "A" was used for that purpose only, the machinery being driven from another source. The coal used in Building "B" was for both power and heating.

It will be noticed that, after equating the radiation characteristics of the two buildings, Building "B" shows 25% greater coal consumption than Building "A," which can be accounted for by leakage, engine-room losses and the difference in the boiler steam pressures.

The usual practice adopted for heating and ventilation is to warm the offices in the first stories and basements by a plenum warm-air system, the temperature of the air being controlled by thermostats. All halls and offices above the first floor are warmed by direct radiators under the windows, with little or no attempt at artificial ventilation.

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The tendency to crowded population has forced the study of artificial ventilation, a satisfactory solution of which has its difficulties, the principal one being the location of the flues in such a manner as will encroach least upon the floor space and at the same time permit changes in the partitions for the modification of offices. These conditions force the ventilation flues upon the exterior walls, or under the ceilings, and the possibility of such locations depends upon the architectural treatment of the building.

When there is sufficient wall space between the windows for the location of exhaust ventilation flues, a satisfactory method is to provide sufficient direct radiation under the windows to supply the loss of heat, due to wall and window exposure, and supply warm air from the corridors to the offices through the transoms, exhausting the air through the exterior wall flues by fans in the attic. The air in the corridors can be warmed by direct-indirect radiators under the windows, or by a supply of warmed air from fans in the basements, and can be kept pure by ventilating the toilet-rooms through the seat vents of the water-closet bowls (operated under a stronger draft than the office ventilation).*

Each office may be supplied by fresh warmed air and heated by direct-indirect radiators; the fresh air is carried in a duct to each radiator from an opening under the window sill, and the exhaust ventilation maintained by flues and exhaust fans.

Some direct-indirect radiators have a base casting provided with a damper arranged so that the air may be supplied to the base of the radiator either from the outside or from the room, or the damper may be set so that part of the air comes to the radiator from the room and part from the outside.

By this arrangement, when the air from the outside is entirely cut off, the radiator acts as a direct radiator and saves steam at night or when the office is unoccupied.

In another form, which is really an indirect radiator, the radiator castings are entirely enclosed in a casing provided with a register, and the fresh-air flue is controlled by a damper. This form is more expensive in construction, with the disadvantage that the room must at all times be heated with warm air, whether occupied or not.

* This method was adopted by the writer in the Drexel Office Building in 1866, and subsequently in the Reading Terminal Station, the Professional Building, the Drexel Institute and the United Gas Improvement Company's Building, with marked success.

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In both forms the steam valves should be under the control of the tenant.

One trouble with this method of heating is that during high winds more air than is required may be admitted, which results in cold drafts. To prevent this, the fresh air under the windows should pass through a tortuous flue to the radiator.

A plenum hot-air supply for the entire building, with the registers controlled by thermostats, has also been used. This method is expensive in construction and operation, it is difficult to control, and the supply of air depends upon the temperature of the room, and not upon the ventilation requirements. For structural reasons warm air is supplied to the offices from the interior walls, which, when sufficient heat is admitted to warm the offices near the windows, results in high temperatures near the heat supply, and, during windy weather, on the lee side of the building.

In the earlier installations, individual radiators were furnished with hand-controlled valves on both the supply and drip-connections with automatic air-valves; the water of condensation returning to the hot well by gravity.

Experience in the use of this system was unsatisfactory, due to failure in the operation of the air-valves and ignorance or neglect, on the part of tenants, in the operation of the controlling valves. To prevent the use of high steam pressures and the consequent loss in engine performance, large supply and drip-pipes were used, which, being difficult to conceal, were generally exposed.

Modern practice dispenses with the air-valve and the hand-operated drip-valve, and reduces the size of the pipes by producing a partial vacuum in the drainage system and in the radiators when not in use, so that a flow of steam to the radiators can be maintained below the atmospheric pressure. In the "Webster System" a thermostatic valve takes the place of the hand-operated drip-valve and the air-valve.

An equal distribution of steam at low pressure, through small pipes, can be readily accomplished by supplying the risers from both basement and attic. The writer has installed such a system, combined with the "Webster Vacuum System," in a 12-story building in which each of the steam supply risers was only 1½ ins. in diameter, and served 600 sq. ft. of radiation. The steam supply to two radiators, serving 54 sq. ft. of surface, did not exceed ¾ in., and the drips from the individual radiators ½ in.

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The amount of steam supplied to the entire system, serving an equivalent of 15 000 sq. ft. of radiator surface, was controlled by a pressure-regulating valve. The temperature of the steam in the radiators under test was regulated to vary from 190 to 227 degrees. The maximum difference in temperature between the initial steam in the boiler-room and the farthest radiator did not exceed 3° when the outside temperature was 7°, and the maximum difference was 1½° when the outside temperature was 31 degrees.

The pipe sizes in this system were about one-half the areas adopted in vacuum systems and one-fourth the areas in gravity systems as laid out by the usual method of supplying the risers from either top or bottom.

Using this method of distribution with the "Webster System," a 24-story building could be served with steam supply risers not over 2 ins. in diameter, and serving from 1 200 to 1 300 sq. ft. of radiating surface, with a better steam distribution than can be obtained by larger pipes and serving only from one end.

The double-supply system decreases the difficulty in concealing the pipes, decreases the cost of construction in tall buildings, and puts the entire installation under more complete control.

In the one-pipe system a single riser is used both to supply and drain the radiators. The risers are supplied at the top, and drained at the bottom. With tight valves and reliable air-valves this system is fairly satisfactory; but the steam supplied to the radiators is wet, the risers are large, and it is difficult to conceal the pipes and provide for expansion.

Lighting.—The safety, cleanliness and efficiency of the electric lamp, added to the ease of distribution and production in the building, make it the cheapest form of artificial light. The standard is the 16 candle-power incandescent lamp, which is used in offices, small rooms, narrow corridors and in places where it is desired to conceal the lamps. High and large hallways and offices are frequently lighted with the enclosed arc lamp.

For general illumination in halls, except where special effect is required, one 16-candle lamp will serve from 700 to 1 000 cu. ft. of space, or, with the usual ceiling height, one lamp for 80 to 100 sq. ft.

In offices the number varies from one 16 candle-power lamp for 20 sq. ft. of floor space to one for 50 sq. ft. of floor space; the average in well-illuminated buildings being one lamp for 30 to 35 sq. ft.

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Outlets are distributed in the offices, so that lamps may be installed for general illumination, and socket connections provided for desk lights and fans.

The writer has found that the maximum amount* of current used will vary from 50 to 60% of the total installation, including the engine and boiler-room lighting.

The general practice is to use direct current at 110 volts, distributed on the two-wire system. In order to obtain a supplementary supply from the public street service some buildings are wired on the three-wire system.

Improvements in incandescent lamps enables us now to serve buildings with current at 220 volts, so that at present there is no necessity for the three-wire system. This voltage is more convenient for electric motors, and reduces the cost of wiring. Its use has been adopted in a number of large buildings.

The wiring layout of a building, to obtain equal voltage throughout, and to provide for easy repair and ready control, is always an interesting study, and each case must be treated separately.

For ample power, a good rule is to provide one I. H.-P. for from ten to twelve 16 candle-power lamps or their equivalent.

Without the installation of an electric storage battery, all the power required in the building, except that for the passenger elevators, may be operated directly from the electric generators serving the light, so that the electric generators may be designed to serve the maximum lighting demand and all the power exclusive of the elevator service.

A high-speed, single-cylinder, steam engine and electric generator, properly rated, will give economical results from 50% under-load to 50% over-load. A high-speed, compound, non-condensing engine will give economical results at half-load. If two units are provided, each rated for one-half the maximum, a 25% load can be operated without serious loss.

Three units, each rated for one-half the maximum load, provides ample power—with one unit in reserve—and can be operated with economy during from 12 to 15 hours.

An additional small unit should be provided to furnish light at night in the engine and boiler-room and for police lighting throughout the building.

* The average maximum amount.

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Water Supply and Sewage Disposal.—The water supply may be divided into four separate heads:

1st.—For the supply of boilers;

2d.—For wash-basins, slop-sinks and drinking purposes;

3d.—For flushing water-closets, etc.;

4th.—For use in the refrigerating apparatus.

The water supply for boilers can be taken directly from the city supply without previous pumping, and the amount depends directly upon the boiler horse-power.

In the latest office buildings each office or suite of offices is furnished with a stationary wash-stand with hot and cold-water supply; a tap is supplied which furnishes water artificially cooled for drinking purposes. Similar wash-basins are furnished in toilet-rooms, and slop-sinks are provided on every floor with hot and cold water for cleaning the building.

The water for this purpose will approximate $\frac{3}{10}$ gall. per day per square foot of office space. The water for flushing water-closets, etc., amounts to about $\frac{4}{10}$ gall. per day per square foot of office space, and the water used for refrigerating purposes will approximate $\frac{3}{10}$ gall. per day per square foot of office space. If ground-water can be obtained for the flushing and refrigerating systems, the bills for water service will be materially reduced.

In most cities, all the water, except that used for boilers and the lower stories, must be pumped; the water is pumped directly into the system or into reservoir tanks in the attic.

In high buildings the water supply may be divided into separate direct-pressure systems serving different levels, or the tank pressure may be controlled by valves.

Filters are installed where the city water supply is inferior, and separate tanks and pumps are provided when ground-water is utilized.

Waste-water and sewage above the line of the street sewers is discharged directly into them, and the drainage and sewage below the street sewers is generally raised by pneumatic pressure to flow into them.

The writer has, by the use of a small septic tank, utilized an ordinary pump, driven by an electric motor, for this purpose, occupying less room and at less cost than the ordinary method.

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Fire Protection.—Although most of the large office buildings now constructed are made as nearly fire-proof as possible, they contain large quantities of combustible material, and it is all-important that a fire should be extinguished before any considerable headway has been gained. Every office building should be provided on each floor with portable chemical fire-extinguishers, and also with fire-hose and nozzles, having an immediate and ample supply of water under pressure, and with electric signals to the engine-room.

To supply water for fire purposes, fire-mains are run through the building and supplied with water from the attic tanks controlled by check-valves, and from a fire-pump in the engine-room. Connections are also provided on the outside of the building, controlled by check-valves, for the municipal fire-engines.

With this apparatus and an efficient watchman service, the risk from fire is reduced to a minimum.

Transportation.—Without an efficient elevator service the office building is a failure; yet, notwithstanding the many elevators which have been installed, the elevator service causes the operating engineer and the manager of the building more anxiety than any other single item of the installation.

Each tenant of the building must be carried at least four times a day, and provision made for the outside public. The transportation should be as rapid as possible within the limits of safety.

The number of passengers carried in some large and densely populated buildings amounts to from 10 000 to 20 000 per day.

In the Drexel Building, Philadelphia, 10 stories high, and having 120 000 sq. ft. of office space above the first floor, between 1 500 and 1 600 passengers were carried in one hour.

After the building is completed it is always difficult and sometimes impossible to modify the elevator installation, so that the proper number and size of the elevators is an important question to determine.

The speed and size of the car, the height and office floor-area of the building and the method of operating must be considered. In tall buildings where more than four cars are installed, the service may be improved by running some of the cars to stop at all floors and others to run "express" beyond the lower floors and "accommodation" above.

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TABLE No. 2.

Building.	Stories.	Office area above first floor.	No. of cars.	Square feet per car.	Area of car, square feet.
St. Paul Building, New York	25	88 200 sq. ft.	6	13 200	23.6
Empire Building, New York	21	150 000 "	10	15 000	42.0
N. American Building, Philadelphia	18	90 500 "	5	18 100	27.6
Real Estate, Philadelphia	17	155 650 "	10	15 560	23.7
Bowling Green, New York	16	223 000 "	9	24 700	....
Land Title, Philadelphia	15	66 400 "	5	13 200	29.6
Stephen Girard, Philadelphia	13	67 000 "	4	16 750	29.0
Drexel Building, Philadelphia	10	130 000 "	6	21 700	21.4

The writer has attempted to formulate a rule, based upon the operation of elevators in various buildings, by taking for factors in the calculation the office floor-space, the number of trips per hour and the size of the cars, which is subject to modification for "express" service.

An idea of the practice in elevator installations may be obtained from Table No. 2, which gives the story heights, the approximate area of office space above the first floor, the number of cars, the office area per car and the area of each car.

To determine the number of trips and car travel per hour, observations were made at four office buildings in Philadelphia:

Drexel Building, 10 stories high;

Stephen Girard Building, 13 stories high;

Land Title and Trust Building, 15 stories high;

Real Estate Trust Building, 17 stories high.

In the Drexel Building, which has 6 elevators, one elevator ran from the first to the fourth story; one elevator from the first to the fifth story; 2 elevators from the first to the tenth story, stopping, if required, at any of the floors, and 2 elevators ran to the fifth story "express," and served all the stories above.

At the Stephen Girard Building there were 4 elevators, which ran "accommodation" through.

At the Land Title and Trust Company's Building there were 5 elevators which ran on schedule time, "accommodation" to all floors except the second.

At the Real Estate Building all the cars ran "accommodation" through.

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TABLE No. 3.

Building.	Stories.	Height, feet.	Travel per trip, feet.	Trips per hour.	Average feet per minute.	Miles per hour.
Drexel Building.....	4	40	80	60	80	0.910
"	5	50	100	52	87	0.984
"	10*	108	216	25	126	1.431
"	10*	108	216	25	126	1.431
Stephen Girard Building....	13	150	300	30	150	1.704
Land Title Trust Building..	15	180	360	27†	162†	1.840†
" " ..	15	180	360	24*	144*	1.636*
Real Estate Trust Building.	17	200	400	25	167	1.894
From these data the following are estimated:						
Bowling Green.....	16	190†	380	26	164	1.87
Empire.....	21	250†	500	22	188	2.08
St. Paul.....	25	300†	600	20	200	2.273

* Actual. † Estimated.

Table No. 3 shows the results obtained, and also an approximate estimate of the running time for the Bowling Green, Empire, and St. Paul Buildings, New York City.

From observations at the Drexel Building in 1897, during the noon hour, the up-travel from the first floor reached 800 passengers, with a maximum of 12 to 13 passengers in the car. The cars were also overtaxed all day, from 10 A. M. until 4 P. M.

At the Land Title Building, running 24 trips per hour, the service was very satisfactory. There was a slight crowding during the noon hour; this, however, can be remedied by the use of the improved car-signalling apparatus.

At the Stephen Girard Building the cars are crowded during the noon hour, and also between 3 and 5 in the afternoon.

All three of these buildings are well filled.

The Real Estate Building is not fully occupied, so that the elevator service there cannot be fairly judged. At present the installation is very satisfactory.

Using the trips per hour as observed and estimated, and equating the car area by the formula:

$$a = \frac{A}{T \times 22}$$

in which a = Square feet of car area;

A = Square feet of office area;

and T = Total trips per hour.

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TABLE No. 4.—Equated Car Areas.

Building.	Stories.	Number of cars.	Square feet of office area per car.	Actual car area, in square feet.	Equated car area, in square feet.	Estimated trips per hour.
St. Paul	25	6	13 900	23.6	31.5	120
Empire	21	10	15 000	42.0	31.0	220
Real Estate	17	10	15 500	23.7	28.3	250
Bowling Green	16	9	24 700	....	43.1	234
Land Title	15	5	13 300	....	22.4	135
"	15	5	13 300	29.6	25.	120
Stephen Girard	13	4	16 750	23.	25.4	120
Drexel	10	6	21 700	21.4	28.2	210

At the Drexel Building, the two cars which ran "express" to the fifth floor, and the two cars which ran "accommodation" through, made the same number of trips, and carried practically the same number of passengers. It would be interesting to know whether similar results are obtained in any other buildings, and the advantage gained in arranging the travel of the various cars to serve "accommodation" through the entire trip, "express" part way, or providing separate service to different heights in the building.

Using the same formula, and equating to obtain the square feet of office space per car, with given sized cars, and the number of through trips heretofore used, we have the following results, with cars of 25 and 30 sq. ft. area.

TABLE No. 5.—Square Feet of Office Area Per Car.

Stories.	Car area = 25 sq. ft.	Car area = 30 sq. ft.
25	11 000 sq. ft. per car.	13 200 sq. ft. per car.
21	12 100 " "	14 500 " "
17	13 750 " "	16 500 " "
16	14 300 " "	17 160 " "
15	14 850 " "	17 820 " "
13	16 500 " "	19 800 " "
10	19 950 " "	23 100 " "

The style of elevator will depend upon the kind of power adopted, the space which can be provided for the operating machinery, and the location of the elevators in the building.

The lifting machinery is driven either by electricity or by water under pressure.

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Electric elevators are operated by a motor revolving either a drum carrying a hoisting cable, or a screw with a traveling nut connected with a system of multiplying sheaves and cables.

In another type of electric elevator two independent motors are connected by endless ropes which run over traveling pulleys suspended from overhead sheaves, the hoisting cable being connected with one of the traveling pulleys. The direction and motion of the car is regulated by varying the speed of the driving motors.

Hydraulic elevators are operated by water from a tank at the top of the building, or from a pressure tank containing compressed air, or by pressure obtained by weights. The car is moved either by a vertical ram or hoisted by cables attached to the car and running around overhead sheaves to pistons in horizontal or vertical cylinders; the length of the cylinders being reduced by multiple power transmission in the hoisting cable.

The elevators descend by their own weight and the cars are counterbalanced so that they may descend empty at the required speed.

The water is pumped either directly by steam engines or by electric motors.

The apparatus should be as simple in construction as possible, to obtain speed and efficiency, with economy in power and cost of repairs, and should be so devised that the failure of the principal parts of the machine, or even of some minor part or safety device, will not be the cause of accident.

Increased economy in power by complicated mechanism may be more than counterbalanced by the cost of the machine and by repairs and supervision.

To provide against accident, each hoisting machine should, whenever possible, have a speed governor to prevent the car from running faster than the maximum required running speed, and, in addition, the car should be provided with independent automatic safety clutches to clamp the car to the guides should the speed governor fail or the car fall. The clutch should stop the car gently, without jolt or jar, and should be under the control of the operator.

The power-operating machinery should be automatically cut off and stop the cars gently at the top and bottom landings.

The overhead sheaves and beams should be sufficiently high to prevent the counterweights striking them when the car is at its lowest

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limit of travel, or to prevent the car from striking when at its highest limit of travel. Clearance should be provided at the lowest landing, of sufficient depth to prevent the car striking when at the lowest limit of travel.

A device should be provided to prevent the opening and closing of the hatchway doors, except when the car is at the landing, so arranged that the doors must be closed before the car is put in motion and opened only when the car is stopped.

The hatchway should be kept clear of all operating machinery, and the machinery should be placed where it can at all times be conveniently inspected and repaired.

The cars in electric elevators are controlled by electrical contact; in hydraulic elevators by either a hand-wheel or lever operating the starting valves. The lever arrangement requires a pilot-valve which controls a small hydraulic motor operating the main valve. The lever and pilot-valves reduce the labor of the car attendant, but add extra mechanism to the apparatus.

The car doors are operated either directly by hand or by a pneumatic apparatus controlled by the foot or by the starting lever.

Where a large number of elevators is installed, a successful signaling device, operated by electricity, is in use. Two push-buttons only are placed on each floor; one signaling the cars going up, and the other those going down. When the push-button is pressed a light is shown in the next arriving car and notifies the attendant in time to stop at the proper landing. Other lights are located over the hatchway door, notifying the passenger, a white light for the up trip and a red light for the down trip.

The hydraulic elevator, unless complicated with a differential water supply, uses the same amount of power for light as for heavy loads.

The hydraulic ram elevator is limited in speed and range.

The electric elevator uses power in proportion to the load, and is more economical than the hydraulic machines.

The objection to this form of elevator has been the difficulty in controlling the large starting current, which makes the lamps fluctuate when the elevator is operated from the same source of electricity as serves the lamps.

With this defect remedied, all the machinery in the building, including that serving the electric light, may be driven by electricity from the same source.

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In addition to the transportation of passengers from the street to the various floors, special elevators are frequently installed to serve several stories occupied by the same tenant.

Provision must also be made to carry freight and heavy safes to all the floors.

Elevators must also be provided for furnishing supplies to and removing ashes and wastes from the basement.

Communication with the Outside World.—Modern business methods require that each office shall have telephone and telegraphic service, so that, except when personal contact is deemed important, all business can be transacted without leaving the building, and systems of conduits for telephone and telegraph wires must be installed.

Management.—The financial success of the office building depends largely upon the economic management of the mechanical plant.

To this end the necessary instruments for measuring fuel and water supplies, for indicating and recording the power produced and utilized and for detecting defects in the apparatus must be furnished.

The building should also have ample storage for coal, ashes and general supplies and a repair shop, equipped with machine and hand tools, supplies of materials and spare parts of apparatus so that ordinary repairs can be made by the employees.

However perfect the installation, without proper management, it will fail; success depends upon the operating engineer, who should be an educated engineer and a cool executive, with ready wit, willing hand and untiring energy, as well as a competent mechanic—the most important machine in the installation.

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DISCUSSION.

ROBERT C. CLARKSON, M. Am. Soc. C. E. (by letter).—The modern office building has been compared to a city or town, and to the human frame, and had the latter been the human body, the simile would have been most apt, for in the human body dwells each being individually, and in that structure each being takes intense interest, devising means to strengthen and repair any inherent or temporary weakness and to keep the same in a complete condition to resist the passing storm or the inevitable decay of time. Mr. Clarkson.

The structural engineer has to do with the skeleton or framework upon which the architect rears his flesh and beauty of wall and ornament, but to the consulting mechanical engineer is given the duty of caring for the brain, lungs, heart, veins, muscles and sinews which give to the whole its mechanical life and fit it to man's uses and needs.

It is to the creation and design of the actuating mechanisms that this interesting paper has been devoted, and the writer deems it fitting to add his chapter to the volume.

The surest method of proportioning the amount of boiler power to be provided is to calculate and construct diagrams or "load-lines" of the various machinery, extending them over a range of twenty-four hours, during the several seasons of the year, to meet the most severe conditions; a combination of the several load-lines, of the machinery to be operated at the same time, giving the maximum power to be generated. The amount of reserve power must be determined by the method of operation, available space, and special conditions to be overcome in each installation.

The heating plant, no matter how carefully calculated and designed, will often seem to be inadequate during the first winter of operation, the greater portion of its work being the consumption of coal to evaporate the moisture from the walls, floors, etc., due to the necessary use of a great quantity of water, and the soaking of rain, during the course of construction.

Ventilation, although not usually so considered, is one of the most important items of office-building construction. The writer, after twelve years' residence in a building* having ventilation for each office and other room, has just completed a year's residence in an unventilated building.† The presence and operation of the ventilation system in the first building was not particularly noticeable, but its absence in the second building is extremely noticeable, and there is a marked difference in favor of the first.

The ventilation system should not be made a part of the heating system, neither should the heating of the building be dependent thereon, otherwise it is necessary to operate the ventilating system at

* Drexel Building, Philadelphia, Pa.

† Stephen Girard Building, Philadelphia, Pa.

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Mr. Clarkson. all times, whether the building is occupied or vacant, with the consequent wasting of coal due to drawing cold air from out-doors, heating it, and distributing it throughout the building.

The toilet-room ventilation is best accomplished, and the most effective results are obtained, by removing the air from the room through the rear portions of the closet and urinal stalls. This method causes a constant flow of air toward the source of odor, thus preventing its dissemination throughout the general room. The registers also present a neater appearance, with a chance of a larger and more efficient outlet than could be obtained by the method of connecting directly to the china fixtures.

The toilet-room ventilation should be kept entirely separate from the general ventilation of the rooms and hallways, the draught in the main flues and ducts being controlled by separate fans or aspirating coils.

It is necessary to make provision for the maximum operating load for electric lighting and electric power, and this is a variable quantity, dependent upon the liberality of the installation and the particular tenantry of the building. The writer believes that the maximum load of the average installation should be 85% of the installed fixtures.

The "proper rating" of an engine and dynamo varies with the viewpoint of the several parties interested in the machinery, and the consulting engineer must determine, from the conditions to be met, the particular sizes and number of units most advantageous to the installation. It is obvious that machinery working ordinarily to its maximum capacity cannot carry 25 and 50% over-load, and machinery to meet such additional calls must be installed, under ordinary conditions, to operate that much below the maximum output. If, however, the "proper rating" is taken as the point of greatest economy of the engine, and the heating limit of the dynamo is kept within the range of 35° Cent. above the surrounding atmosphere, the conditions of 25 and 50% over-load for varying periods can be taken care of without undue waste in operation or risk of damage to the machinery.

The conveying and disposing of sewage from fixtures below the sewer level can be accomplished with less risk of stoppage, and more ease of operation, if the motive force is compressed air applied directly to the vessel holding the mass to be disposed of, rather than by the use of a pump operating from a tank. By the use of compressed air the working parts exposed to corrosion and clogging are reduced to a minimum, and the operation of the machinery is cleanly, and with facility for repairs.

The matter of fire-protection is a problem also contingent upon the uses and tenantry of the building. The installation of fire-extinguishers is not always to be recommended, for, in many instances, they are allowed to become neglected and inoperative, while, at best, few per-

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sons are aware of the nature and proper use of such appliances. If fire-extinguishers are installed, they should be given frequent attention as to fittings and contents, and they should be accompanied by directions as to their proper use printed in bold letters. Mr. Clarkson.

The installation of a vertical fire-pipe, with hose at each story, is the usual method of fire-protection, but this is seldom erected with complete facilities. The top of the line should be connected to a tank at the highest point of the building, in such a manner that the line is at all times filled with water, and the bottom of the line should be connected to the largest pump or to a special fire-pump in the machinery room, with a side connection, outside the building, for the connection of the apparatus of the city fire department. With proper signaling apparatus from each hose outlet directly to the location of the pump, and the introduction of the necessary check-valves, the fire-line should be always ready for instant service, with a possible supply from three sources, varying as to the extent of the fire.

The use of automatic hose-reels, or those in which the revolving of the reel opens the valve and turns on the water supply, is not to be recommended. More damage can result from the uncontrolled water than the fire can accomplish in the few seconds necessary to turn a hand-valve. The use of sprinklers in office buildings is also inadvisable, for the same reason,

The most effective fire-protection is careful watchmen, with regular fire-drill of all employees.

The elevator service should be considered as a vertical street-car system, the hatchways and guides corresponding to the streets and rails; and the operation of the cars must be considered upon similar lines. It would be manifestly impossible to operate a street railway, with any degree of efficiency and carrying capacity, if the cars were expected to run at such speed and under such control that a stop was possible every 12 ft., yet this is expected of an elevator service where the cars stop at each floor.

The problem may be solved by dividing the building horizontally into sections, and providing for each section cars of such size and number as will accommodate the average maximum number of passengers from each, with the addition of two or more cars to stop at all floors of all sections.

The use of such an arrangement requires a certain education of the public, but, with the tenants as guides, the outside public soon learns the special arrangement of each particular building.

The elevator cars and enclosures should be arranged to give the greatest facility of loading and unloading. This is best accomplished by double gates, opening the whole width of the car, and by the use of signalling apparatus directing the operator when two or more stories away from a desired stop. If mechanism is introduced to operate the

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Mr. Clarkson. doors, it should be controlled separately from the mechanism controlling the movement of the car, and the equipping of doors and gates with devices to prevent the starting of the car until the gates are firmly closed and fastened should be insisted upon.

The area of the cars should not be less than 5 by 5½ ft. and the maximum speed should not exceed 500 ft. per minute, for the safety devices must be set to operate at a speed greater than that of ordinary running, and at a running speed of over 500 ft. per minute the car will acquire too much momentum before the devices are brought into action, which may result in serious damage, even though all known safety devices are in use; this is particularly the case if the operator should lose control of the car near the bottom of the hatchway, unless the hatchway is fitted with an air-cushion.

The question of motive power for the elevator system admits of many varying answers, and the condition of operation is an important factor in reaching a conclusion as to the most advantageous power. With hydraulic power, the question resolves itself into the method to be used.

One of the smoothest running forms of hydraulic elevator is that of the direct ram or plunger type, which, though there are limits to its use, is available for heights of 200 ft. and speeds up to 600 ft. per minute. This form of elevator combines safety, with economy of operation and repairs, and minimum space occupied.

Another type of hydraulic elevator is the vertical cylinder, which is smooth running, and economical in space, but care must be taken to provide means to remove the accumulations of air from the cylinder, otherwise the car will rebound at each stop.

The type of hydraulic elevator probably in greatest use is that with a horizontal cylinder, and while not economical in floor space, it has many advantages to commend its use, its capacity being limited only by the length of the cables obtainable and the ability to construct cylinders of the desired length. With this form of elevator care must be taken to provide means to support the long horizontal cables when the piston is at its maximum extension, otherwise the car will rebound if suddenly stopped.

Where the conditions are such that the use of hydraulic power is impossible, the electric elevator is available; but the electric elevator requires constant attention, and successful operation depends upon the contact of many hundred small wires, any one of which may become loose and disable the apparatus.

If electric power is more accessible than steam, the combination of an electrically driven pump with a hydraulic elevator makes an installation economical in operation and combining the economy of electric power with the simplicity of the hydraulic cylinder.

Other items of lesser importance, such as distribution of cooled drinking water, filters, telegraph and telephone communication, time

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distribution, watchman's clocks, etc., vary so much that they must be solved to suit each structure, and are not amenable to any fixed rules. Mr. Clarkson

In all the foregoing it is supposed that the architect will co-operate with the engineer, and provide ample space for the installation of all necessary machinery, and that the owner will provide a sufficient number of skilled attendants to operate and care for the apparatus. Many times, however, does every engineer find that he must, to save his reputation, either antagonize the architect, to obtain sufficient space, or the owner, to obtain permission to install machinery of sufficient size and construction, or is forced to abandon the work entirely; every foot of space devoted to machinery, ventilating flues, etc., being deemed as so much loss to the money-earning value of the building.

JOHN W. HILL, M. Am. Soc. C. E. (by letter).—This paper and the discussions contain enough matter to show that the mechanics of the modern office building are sufficient to constitute an art, and require for their proper treatment the aid of a specialist in this line of mechanical engineering. With reference to the steam heating of the tall office building, several factors seem to require consideration: Mr. Hill.

(1) The volume or cubical contents of the building.

(2) The exposure of dead wall surface and glazed apertures.

(3) The ceiling height of stories.

(4) The construction of dead wall (whether solid or with defined air-spaces).

The minimum temperature of the atmosphere for a given locality, and the maximum velocity of winds to be considered, must also enter into any formula which undertakes to provide mathematics for the steam or other methods of heating.

The influence of light-wells and elevator shafts in forming an upcast, and of leaks around door and window frames and sash withdrawing warm air from the surrounding rooms, must have an effect on the cost of heating, although the ventilation thus afforded should be considered as an advantage to the occupants of the building.

The indirect system of heating, *viz.*, by drawing fresh air from the outside of each room or floor around the steam radiators distributing the warm air through the rooms and withdrawing it through ventilating pipes or ducts, has the advantage of frequent change of the air inspired by the tenants, but in the matter of coal charges will usually be the most expensive system of heating to maintain. There is no good reason why this system cannot be successfully operated, but the cost of introduction and operation doubtless opposes its more extensive use in modern public buildings.

High fuel economy cannot be expected in the power required to operate the elevators, when, within a few seconds, the loads vary from no load at all to a maximum; but some system which will admit of stor-

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Mr. Hill. ing energy during the periods of low work, to be given out during periods of high work, by storage batteries, water tanks or air receivers, should give the best economy, although such systems will not be adapted to handle rapidly heavy loads quickly repeated. Hydraulic elevators, arranged to consume water in proportion to the load handled and the distance traveled, should represent the least unavoidable loss between the development and application of power. Whether such elevators have been reduced to practice the writer does not know, but efforts in this direction were made more than twenty years ago. Such a system, if supplied from the city mains, should be an economical elevator power, provided the water rents are moderate. Electric elevators supplied from "commercial" currents are economical in consumption of power, and should consume current approximately in proportion to the load handled and the distance traveled. Local conditions must control in each case, and no rule of universal application can be laid down for the source of elevator power. The fact that an electric current is needed for illuminating the modern office building, and that this current can usually be supplied more cheaply in the building than by connection with "commercial" currents suggests the further development of the electric system to provide power for the operation of the elevators and other motion machinery in the building.

There are one or two features of the modern office building, not of an engineering nature, which possess interest alike to the investors and patrons of such structures, *viz.*, the financial returns upon investments in such enterprises, and the economies and advantages to the occupants.

While the financial returns must necessarily vary with the value of ground and improvements, and the location, and with the character of the business conducted by the tenants, still it has happened that thus far such buildings are promptly and steadily tenanted by people who rarely default in their rents; and, upon the assumption that the owners have proportioned the rents to the gross investment and maintenance charges, it is fair to believe that such buildings are usually profitable sources of investment. At the same time it would be of general interest to compare two columns of figures, one containing the interest, and other fixed charges due to the invested capital, and the operating and maintenance charges, and the other the gross income from rents. Some of these buildings, as is well known, have cost more to construct than certain prosperous short lines of railway, and many of them have been built by syndicates or corporations organized in quite the same manner as railway corporations.

The fact that such buildings find tenants, even before they are ready for use, suggests that the tenants find the use of such buildings profitable, or are willing to pay for the extra comforts and conveniences which they afford.

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Contrasting the old-fashioned office quarters, with their poor light and heat, and their generally uninviting appearance, with the modern conditions, suggests that, aside from the contracted space which is now often made to answer for formerly large rooms, there are advantages to tenants for which they are willing to pay, and for which some benefits are received or expected. Mr Hill.

The general construction and finish of the rooms in the modern office building enable the tenants to dispense with certain accessories of the old-style quarters which represent some gain and much comfort. Steam heat, lavatories, vaults, telephones, mail chutes and elevators save money or time, and their maintenance by the owners of the property relieves the tenant of much annoyance and labor which was his under the old system.

In some respects there is a strong resemblance between the modern office building and a Pullman sleeper. In both, the floor space is more contracted than one desires, but this inconvenience is partly, if not wholly, offset by the conveniences which accompany each. In both, the effort is made to satisfy all the usual and probable requirements of the occupants with the least sacrifice of time and space.

The assembling in the modern office building of many businesses of like kinds promises a distinct advantage to both the user of the building and his patrons. Thus, "The Rookery," in Chicago, for example, contains a collection of offices largely interested in the production or use of iron, steel and similar materials, and of industries akin to these.

Two things, however, contribute essentially to the commercial success of the building; the high-speed elevator and the telephone. Indeed, the elevator is the germ from which the sky-scrapers have sprung; without the elevator such buildings would be impossible; and, as the needle with an eye in the point is the feature of the sewing machine, so likewise the modern high-speed elevator may be said to provide the commercial foundation for the modern office building.

The writer's experience with modern office buildings has been that of a tenant, and, from this point of view, it is surprising with how little space one can conduct his work when he has to, and how much of the litter about an engineer's office can be neatly and compactly stowed away and held for future reference if space is cramped. The evolution in business conveniences is not limited to the well-lighted and well-heated office building, with high-speed elevators making the upper floors almost as desirable as those lower down, but is carried into the necessary furniture of such buildings. Compactness and directness characterize all steps taken to develop and furnish the modern office, and this underlying principle doubtless has its effect on the mental faculties of the occupant, in suggesting the same use and application of his time and energies as it does of the space and conveniences at hand.

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Mr. Furber. WILLIAM COPELAND FURBER, M. Am. Soc. C. E. (by letter).—Some of Mr. Bolton's remarks indicate that he recognizes, what other keen observers have already recognized, that the high, steel-skeleton office or hotel building is not simply an architectural problem, but a problem in which the architectural treatment is but one of the co-ordinate parts of the design.

The architectural treatment is not and should not be considered as the controlling factor—though it generally is—in the design of this modern embodiment of concentrated facilities for administrative business or communistic living.

In buildings of this character a very little thought will be required to perceive that the architectural design is not the controlling factor, and therefore should not be allowed to fix the conditions, to which all other parts of the design should conform. To illustrate this idea: The architectural design can be changed, almost an infinite number of times, without the mechanical engineering essentials being changed at all, and with only such changes in the structural design as may be required locally by the changed architectural disposition of the materials composing the walls, etc. If the truth of this proposition is recognized, it is evident that no extended argument is required to show that the present method is wrong, in putting in the hands of the individual designer of the façade the final decision on all matters of design; and the truth of this proposition can be admitted and acted upon without in any way belittling or making light of the architectural problems, which are of great importance, and should only be entrusted to broad minds and capable hands. For as long as appearances continue to be a factor in human affairs, and as long as beauty of line and form appeal to our sense of intellectual fitness, just so long will the architectural problem be worthy of the most careful consideration.

The recognition of the fact that the architectural part of high-building construction is not the governing factor gives no excuse for slighting this part of the design, but it should be also recognized that the architectural problem, in its specific relation to high-building construction, can be worked out to a rational solution only when the limits which bound it are defined by the factors which actually should control the design as a whole. High steel-skeleton building construction may be defined as an engineering work with an architectural accompaniment.

It is perhaps already evident that in order that each of the elements entering into the design as a whole be given proper recognition, and its due value as a factor, the chief designer should be a man capable of judiciously passing upon the claims of each of these elements to precedence. Experience has shown that the man who owes his training exclusively to the Art School and its influences cannot or will not con-

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sider the engineering side of the problem. The men who spell their art with a capital A deplore Science, as a necessary evil, which disturbs their mental repose: and, if not as an evil, then as a bore, which interferes with and places practical obstacles in the way of their flights of fancy. Mathematics and Science are looked upon by men of this type as matters of formulas, rule-of-thumb and mechanical routine, a matter to be learned out of a book, which can be mastered in about the same way that one learns to recite a piece of verse or prose, or as one learns a trade. Men of this type do not think that imagination enters into the study or practice of science, and particularly the science of engineering, and look upon themselves as the only rightful possessors of the wings of flight or imagination. They have no conception of the trained imagination required to comprehend the higher mathematics, nor of the sustained and logical derivatives of the product of thought, obtained from mathematical investigation. Having no appreciation of the intellectual basis of engineering, it is not to be wondered at that they treat the demands of Science with scant consideration. It is, therefore, hardly to be expected that the graduate of the Art School only whose whole mental training has been given up to the study alone of the relative values in the play of light and shadow, the balancing of solids and openings, the study of the appearances of things, or the art of seeming, can be qualified to pass upon problems which deal with realities or with things as they are. In works of magnitude, under the guidance of such minds, the structural and mechanical parts of the design are often sacrificed to supposed requirements of Art; and the result is an incongruous composite of the total elements of the design, and, necessarily, a failure. Mr. Furber.

Another type of mind which is unsuited for the position of designer-in-chief is that type which has no appreciation of art and beauty, and has no other "measures" than those of "economy" and "efficiency." This type, while useful in a large way as a counterweight and a standard of comparison, often misses the best solution, or possibly never attains it, in a problem of this character, by the narrow view it takes of the things with which it is not familiar, and, by reason of one-sided training, is not fitted to be an intelligent and impartial judge of the various elements entering into a composite and complex design.

The designer best qualified for this important position of designer-in-chief is evidently one who has a knowledge of the broad essentials and limitations of the various parts of the design, the judicial temperament permitting the balancing of an advantage against a disadvantage, a breadth of mind enabling a comprehensive view to be taken, coupled with courage and decision; with all these qualifications, governed by a broad and catholic love of beauty in all its manifestations—not only of beauty of form and line, which is evident and can

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Mr. Furber. readily be comprehended, but of the more subtle beauty of Science and its results.

This combination of qualities is not an impossible one, or, in fact, necessarily a very rare one. It is realized, perhaps, more definitely in the scholarly engineer, whose acquaintanceship with facts and laws governing matter is accompanied with a broad knowledge of the fundamental requirements of Art, and to him must eventually come the position of Master Builder.

If the composite design of one of these engineering-architectural problems is analyzed, it will be seen to admit of two main subdivisions:

(1) The engineering design;

(2) The architectural design.

The engineering design can be subdivided into its structural and mechanical components: The structural component, comprising the foundations and superstructure, including the stability of the structure against vibration, wind forces, fire and corrosion. Under the head of structural design should also come the planning of the floor areas and spaces, with all their subdivisions. Usually, the floor plan is laid out by the architectural designer without reference to the structural design, and this results frequently in poor and expensive construction. The consideration of the floor plans and the structural design should take place at the same time—and preferably by the same mind, or at any rate by minds working in harmony, for the floor plan and the structural arrangement are truly dependent on each other. The arrangement of the floor is not strictly an architectural problem, though it is frequently considered as such; it is also a scientific one, or one in which all the factors can be determined and balanced one against the other, and an equation arrived at figuratively, from which a result can be obtained which rests upon determined facts, and not upon mere whim, prejudice or predilection. Daylight has a certain carrying and distributing power which does or should determine some of the dimensions of rooms.

Transportation facilities and convenience of access determine the local arrangement of the office areas around the elevators, or rather the location of the elevators with reference to the "center of gravity" of the office areas. The economic location of lavatory fixtures requires the grouping of these fixtures around common drainage and water supply centers, which in turn demand certain broad arrangements of lavatory rooms and fixtures, usually not difficult to comply with.

The lighting and heating systems require distributive local centers, and sub-centers, which should bear certain relations to the groups of offices and sources of supply.

Not infrequently, the floor plan is decided upon and then turned over to the structural designer with instructions that nothing can be

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changed. This results sometimes in the use of structural expedients which are expensive and structurally unsatisfactory, and which could have been avoided by an appreciation of the relative values of each of these matters. The consideration of the scheme, also, from one standpoint only, often results in the sacrifice of the best arrangement of the mechanical equipment. Mr. Furber.

When the planning of the floors and the structural design is under discussion the fire-proofing and rust-proofing should also be considered. It is not an uncommon thing to find the fire-proof covering cut away to get in a door or window frame, or some piping that has been overlooked or not considered when the planning was under consideration, and for this there is no excuse, unless haste in preparing the plans —due to no fault of the designer—is given.

Fire-proofing is generally given but scant consideration, and rust-proofing little, if any, and for this neglect no adequate excuse can be given. The fire-proofing is seldom considered as a part of the design, but is taken care of by general methods of allowances over the dimensions of the beams, girders and columns. The intelligent designing of fire-proofing demands a knowledge of the weakening effects of heat on steelwork. The maximum strength of iron is at about 400° Fahr., and there is a loss of strength of about 1 000 lbs. per square inch of section for every increase of 100° in temperature. A knowledge of the rate of transmission of heat through various substances and dead air spaces, and a knowledge of the resistances of various materials to the action of fire and water, separately and alternately—a little knowledge of these various matters—will be sufficient to show the insufficiency of many of the present methods, and the inadequacy of many of the materials used for this purpose. Porous or semi-porous terra cotta properly made and burned is probably the best material in use, but, as it is manufactured to-day, it is too thin, and the standard beam coverings, with a thickness of only 1 in. over the lower flanges of the beams, are absurd, and the column coverings and the methods of fastening them on the steelwork are but little better.

The question of rust-proofing is seldom considered, and is often entirely ignored. The rusting of iron is such a familiar fact that it is surprising how little attention is paid to it. Millions of dollars are being invested yearly in buildings the durability and stability of which are entirely dependent upon the iron framework, yet, beyond giving the metal-work a coat of paint, nothing is done. The fact that Portland cement preserves iron from corrosion is now well known, and the chemistry behind this fact should be as well known. Rust is caused by three factors working together, *viz.*, water, an acid, and oxygen. If these three factors are not allowed to act co-ordinately, rusting cannot take place. Portland cement, because it is a product of lime, furnishes a base which neutralizes any acid likely to be pres-

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Mr. Furber. ent under ordinary conditions, and therefore any water which may filter through it can do no harm. This fact is acted upon in the use of steel in the foundations of buildings below the water line, but little use is made of the knowledge above the cellar floor.

The mechanical component of the engineering design, comprising the steam power plant for operating the elevators, the lighting, heating and ventilation systems, and the layout of each of these parts; the lighting system, consisting of dynamos, distributive wiring and storage battery; the heating system, consisting of the distributive piping, etc., with the fans and ducts required to supply the heat and air under pressure, and to remove the foul air; the operating mechanism of the elevators; the water supply for individual and general uses and fire-protection; the drainage system, and the telephone and telegraph wire systems, should all be considered and tentatively laid out before the final plans are decided upon.

The requirements of each of these subdivisions of the mechanical design require most careful study, and, as they are a vital part of the design as a whole, they should be laid out to accomplish the best possible results, and this arrangement adhered to as closely as is permissible with reference to other parts of the scheme. Most of the evils of crowded and unsatisfactory layouts of the mechanical equipments in high buildings are the result of lack of knowledge, on the part of the designer-in-chief, of the conditions of arrangement conducive to the best results. The fact that the equipment can be placed and operated in inconvenient and contracted spaces is also taken advantage of, and the mechanical equipment relegated to such spaces as can be devoted to no other purposes. When the amount of money required to maintain and operate this equipment, and its proportion to the total operating expenses of the building, is considered, no defence can be made of the inadequate facilities usually allowed for its installation.

In the distributive systems of the mechanical plant, the placing of pipes, conduits, etc., in the floors, walls and partitions is frequently not considered in the detail drawings, and is generally merely specified. This neglect sometimes leads to a great deal of trouble and often to the adoption of make-shift methods, which could have been avoided by due consideration being given to the details on the drawings.

The architectural design consists in the main of the treatment of the façade, usually according to, or with the details of, some historic style; the arrangement of the windows, and, in a secondary way, the architectural treatment of the offices: This internal treatment of offices can be said to be independent of the general architectural, structural and mechanical design, and therefore not a factor in the design of the whole, with which only this particular discussion deals.

The architectural part of the design can be changed according to

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the whim or predilection of the designer without materially affecting the other parts of the design. It is therefore evident that with this liberty goes great responsibility. If the design decided upon is one of a possible thousand, and the design is a poor one, the responsibility is proportional to the number of better ways of solving the problem. Mr. Furber.

A great mistake that many architectural designers make in treating the high skeleton building is in attempting to treat it as a masonry structure. This lack of perception of the limitations of masonry construction leads them to many curious anomalies and absurd results, as, for instance, the placing of a Greek temple on top of a many-storied building, and the use of brackets, consoles, modillions and other architectural details which once had some function, but which are now, in these buildings, wired or strapped upon an iron framework.

The high skeleton building is worthy of an architectural style consistent with its serious purposes and the dignity of its place in the modern business world, and this style should express the truth of its construction and its purposes, and should not make it masquerade in borrowed clothing, pretending to be a masonry structure, yet violating some of the fundamental principles of masonry construction. Architecture needs the stimulus of new truth and new conditions, and the skeleton building furnishes them.

REGINALD PELHAM BOLTON, M. Am. Soc. C. E.—This paper is somewhat of a skeleton. The author, of course, is dealing with a very wide range of subjects, as an office building is a structure so filled with apparatus that it can be compared to the human frame. Mr. Bolton.

The necessities of the building, in the order of their importance, are somewhat wrongly placed by the author. Heating is, of course, the vital element in these buildings, as in all buildings or domestic structures. Second in importance, however, is transportation, as, without transportation high buildings would be folly. The third is sewage disposal, and after these come matters of convenience, such as ventilation, the supply of ice water, etc. What the author describes as "policing," consists in having an ex-policeman acting as hall man in the main hall, a matter that is not a mechanical subject, and, in New York City, is usually accomplished very easily.

The speaker regrets that he has some contradictions to apply to some of the figures presented in the paper, but, as regards New York practice, they certainly should not go forth as being definite.

To begin with, the author indicates the size of the boiler installation, basing it upon the cubical contents of the building, a most misleading element, as the cubical contents have no relation to the horse-power required. There may be, for instance (and this is frequently the case down town), very tall buildings which are completely enclosed

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Mr. Bolton. by their neighbors. In such cases, the exposed surface is very materially reduced, and the horse-power required by them would bear a very small relation to the contents of the building. The proper way to deal with this matter is to take out the exposed surface and heat losses from the building, and compare them, in extreme weather, with the waste steam which can be provided from the various services that are to be applied, and ascertain whether the exhaust steam will suffice for the purpose of replacing the losses, and if not, what addition should be made to the boiler plant, plus some spare unit or extra amount to cover the exigencies of repair. Some figures relating to tall buildings in New York City will give a better idea of the requirements. The Hudson Building, which is a long, narrow structure, about 60 ft. wide, but nearly 200 ft. in depth, has an exposed side to the north, which will be so until its neighbors are built up. In that building the speaker installed a total of 520 H.-P. The building is about the same size as the German-American Building at the corner of Nassau and Liberty Streets, in which 320 H.-P. proved to be sufficient. In the Hudson Building there are only four elevators, while in the German-American Building there are now six elevators; therefore, the cubical contents of these two buildings bear no relation at all to the amount of boiler plant each required. If the cubical contents are used only as a very rough estimate, about 4 000 cu. ft. might be allowed to a horse-power; but the outside surface of the building is the real element of computation.

The author has not dealt with one very important point which would be of extreme value if brought before the architectural profession in New York City, and that is the subject of chimneys. In the course of the speaker's practice in these buildings he finds that the architects locate the chimneys according to the decorative design or the interior plan of the building, and with no relation to the proper place for the boiler plant; and that they also have a way of asking an engineer the area of the chimney of a given height, and then dealing with that area in any shape that may be convenient to them. The speaker, in one instance, gave the area of a chimney, of which the architects made two flues, and those flues were so narrow that they were almost slots in an outer wall, further complicating that by uniting the two flues at the top, so that the smoke from one boiler went up one flue and backed down the other. Those are some of the trials that engineers have to put up with.

The question of building chimneys in, is one of considerable importance. It is very common now to use steel-plate stacks, and their convenience and desirability is undoubted, but the practice of putting them in flues of practically the same size as the stack, and building them in so as to make them positively inaccessible for all time, is, in the speaker's opinion, utterly wrong. While he at first insisted that

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stacks should be placed entirely outside the building, and so installed them in the case of the German-American and Bowling Green Buildings, he afterward persuaded architects to arrange them in shafts of such size that a man on a boatswain's ladder could be let down the shaft to attend to the outside of the stack. In the Broadway Chambers Building, where the stack is about 240 ft. high, it can be removed in sections from the top, if such should ever be necessary, which may prove to be a useful provision. That is only one of the troubles with which engineers have to deal in the architectural design of these buildings. Mr. Bolton.

In connection with heating and ventilation, the author gives data as to the temperatures in various parts of the country, but, dealing with New York City, he gives the average temperature of the heating season as low as 30 degrees. Through the heating season, it is not as low as that, being 36°, but, of course, when designing a heating plant, extreme conditions must be taken, and in his practice the speaker works from zero, although he has never actually witnessed zero, except in some of the sensational newspapers.

The wind pressure on the building is, as the author states, a very important element, but to take only from 25 to 45 miles an hour as extreme wind speed is not, in this climate, sufficient, in the speaker's judgment, and it is common practice among heating engineers to adopt 60 miles an hour. Under such circumstances, the ordinary office building gives a great deal of trouble from the infiltration of air through new window-casings. All heating plants are tested upon the first season's use of the steam, and at that period, naturally, the windows are at their worst. In the Lord's Court Building, in which the speaker laid out the heating plant, there was at the back a large court, of a peculiar shape, and when the wind blew from the southeast, it came over the top of the building, down into the court, and produced a plenum in the lower part of the court. With an anemometer a speed of 4 miles per hour was measured through a crack in one of the windows of the lower floors, and it was found necessary to take heating surface from some of the upper rooms and transfer it to the lower rooms in order to make them comfortable. In the second season all the windows were supplied with weather strips, a simple precaution, easily applied. In the speaker's practice he makes allowance for leaky windows, assuming a crevice extending around each window admitting cold air at a certain speed, and providing for that air to be heated by the heating surface in each room. It would seem to be a simple matter, however, to make tighter windows than are made now.

At the present time, the speaker has charge of a very large building, which has been under construction for more than a year and a half. The framing of the windows has been in place and has been

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Mr. Bolton. standing without sashes through the spring and summer, and in passing around the other day and looking at the construction he was able to push his fingers between the stone sill and the wooden sill of the window in many places. On remonstrating with the architects about it, he was told by the builder that that was a usual condition, and that it would be plastered up before they got through. That is to say, the only protection received by the radiator, located below the window, is the plastering which is put under the sill by way of stopping such cracks. It is no wonder that considerable steam is required in some buildings for heating purposes, and that the engineer is often greatly disappointed by finding that, from reasons such as the above, and which are not apparent, a good plant, carefully laid out, is using more fuel than he calculated.

The tabulated comparison of two buildings, on page 4, is not definite, because the author has introduced indirect ventilation, which is an element of considerable doubt. In New York City, office buildings are but rarely fitted up with any indirect ventilation at all, and while that may be deprecated as a fault, it still gives a chance to compare directly the heating results of one building with the other. It would occupy too much time to go into details of that nature, but the speaker has records of a number of tall buildings in New York City which are instructive, but even then all the conditions of the services have to be taken into account, and they vary so greatly that it is almost impossible to base very definite conclusions upon them.

Passing to the end of the author's remarks, it may be stated that when everything has been said and done, when every element has been placed in the building that can be arranged for its economy, the operating engineer must be considered, and between him and the coal bill the consulting engineer has at times great difficulty in reconciling facts.

Recently, in the case of a large hotel which the speaker had planned to run with from 7 to 8 tons of coal a day, he was told that they were consuming over 10 tons a day, and on examination could find no special reason for it. He finally suggested that, instead of changing the apparatus, they change the engineer, and this made a reduction of a ton a day.

Referring again to page 4, it is not "the usual practice," in New York City and the surrounding district, at any rate, to warm the lower offices in the first stories and basement by a plenum warm-air system; in fact, it is a rarity. In place of suggesting, as the author does, that indirect radiators be placed under the windows and provided with an air supply from outside, the speaker strongly recommends that this be not done. The wind pressures, especially in respect to tall buildings, are such that there is no possible means of preventing the cold wind from passing through the radiators, and sometimes actu-

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ally freezing the steam in the radiators, and all buildings that have been fitted with these devices, so far as he is aware, have either blocked them up entirely or have met with some such difficulty, as referred to. Mr. Bolton.

In that connection, it seems to be a curious thing that in planning the heating systems of tall buildings the element of height is not taken into account. Some five years ago, the speaker, in making calculations for surface, began using an addition for the height of each floor by adding an amount proportioned on the square root of the height from the street, and in that way a small amount of surface was added for each floor. The result of the usual practice is that on the eighteenth floor, with equal exposures, are found radiators of the same size as on the eighth floor, which is really entirely wrong, because the wind speeds increase so very greatly with the height.

The speaker endorses all that the author states with reference to the benefits of the Webster Vacuum System. Having installed the first one in the German-American Building, now nearly six years ago, he was told at that time that he was doing something that was likely to give him a great deal of trouble, but it has turned out otherwise, and the vacuum system has proved to be exactly the right thing for the peculiar conditions of tall buildings. It has another feature of advantage, on which the author does not dwell. In removing the moisture and condensation from the radiator as rapidly as it is formed, and immediately upon its being formed, it is possible to introduce into the radiator a very small amount of steam, expanded, or wire-drawn, to a lower pressure and temperature, so that the temperature emitted by the radiator can be proportionately reduced. That is done simply by screwing down the supply valve. In order to improve that effect the speaker adopted the system of very small supply pipes to the radiators. In the Grand Central Station, in which this was first done, and where radiators of as many as 100 sq. ft. of radiating surface are supplied with a ¼-in. pipe, it works very admirably indeed. In this way the temperature can be regulated from 90° up to the full temperature of the steam. The speaker has been informed that the system of supplying risers at the top of the building as well as at the bottom has been the subject of a patent, but he is unable to see how that method could be patented. Hitherto, the mere word "patent" has been sufficient to prevent many architects and owners from considering what otherwise would be a very admirable method. In supplying steam to the single-pipe system he has used the overhead, or what is commonly known as the Chicago, system of distribution, in cases where the architects had provided an attic; but it is very difficult to get them to do that, because it makes a small addition to the length of the steel frame, and, of course, adds to the cubical contents of the building, and they begrudge the necessary space. Therefore, in nine

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Mr. Bolton. out of ten modern buildings, when supplied on the single-pipe system, the mains are distributed in the cellar, and present many difficulties by reason of the congested space. It would not be safe in a 24-story building to reduce supply risers to 2 ins. in diameter when serving 1 200 sq. ft. of radiating surface, because, with that extreme height, the probabilities of the upper portion making a heavy draft upon the supply are so great that a 2-in. pipe would be insufficient.

The author remarks that in the one-pipe system "it is difficult to conceal the pipes and provide for expansion." That difficulty is not as great as appears. In the Broadway Chambers Building the supply comes from overhead, the larger pipes being thus above. In bringing them down in the outer walls, instead of using the ordinary expansion joint or spring at the middle floor, they were swung over, concealed under the beams from the position of one riser to that of the next riser, thus providing necessary spring without exposure.

On the subject of electric lighting, the author makes the broad statement that in these buildings "the maximum amount of current used will vary from 50 to 60% of the total installation." There should be a qualifying word in that statement; it should be the "average" maximum amount. It would be totally unsafe to assume that all the lamps in a building would never be used, because that is so in some instances. Sudden thunder storms come up on a summer afternoon, and, while they last, every light in the building is turned on. It is absolutely unsafe to depend upon using only 60% of the total installation. There are buildings in New York City, however, in which the generators, even with 25% over-load, cannot carry more than 60% of the total installation, but for emergencies they depend on a "breakdown" connection with the illuminating company. The usual practice is, as the author says, to use the two-wire system at 110 or 120 volts, but he goes on to advocate the use of high-tension current at 220 volts. The speaker designed the wiring of the R. G. Dun Building for a 220-volt current, and thought the wiring millennium was arriving. The reduction in the cost of copper was tempting, the lamps appeared to be of about the same price, and it seemed to be advantageous to distribute both the motor load and the electric lighting load on the same basis from a two-wire switchboard. There was trouble with the distribution boxes, however, because a fuse letting go at the bottom of it would burn out the whole line. The lamps are inefficient, and, although they are somewhat improved now, they are still not as efficient by nearly 20% as the lamps at 110 to 120 volts; nor do they have so long a life, on all of which grounds the speaker bases his opinion that no one should adopt the higher voltage without serious consideration.

The 220-volt system was also installed in the Lorraine Hotel. The tenants, assuming that the system was about the same as they had usually dealt with elsewhere, attached hair-curling tongs, portable

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lamps, fans, and all kinds of electrical notions, the result being that the fuses were constantly blowing out, which kept the chief engineer busy most of his time repairing breaks and grounds. The tenants in hotels in these days imagine that the current is there for all purposes they may desire. A tenant was recently found driving a lathe with a ½-H.-P. motor connected to a single lamp outlet, and he would not pay anything extra for the current, either. In a hotel in Philadelphia, the owner found his am-meter mounting up in an astonishing way, and, on paying a visit to some of the tenants at an odd hour, found that they had removed the 16-candle-power lamps and had put in 50-candle-power lamps all through their apartments. When it was pointed out to them that the addition was not in their lease, they were very much offended, and some of them left, though he carried his point with the remainder. Mr. Bolton.

It has been suggested that by taking a leaf out of the enemy's book, putting in at first lamps of 10-candle-power, scratching off the number, of course, and afterward, if more light was wanted, by putting in 16-candle-power lamps which are marked 20, the tenants would feel thoroughly satisfied.

The author states, on page 8:

"A high-speed single-cylinder steam engine and electric generator properly rated will give economic results from 50% under-load to 50% over-load. A high-speed compound non-condensing engine will give economical results at half-load."

This phrase is very indefinite. Economical results are those which may be attained under certain conditions, but "proper" rating is a very indefinable element.

The author also says:

"If two units are provided, each rated for one-half the maximum, a 25% load can be operated without serious loss."

These statements are misleading, and, apparently, are written under a misapprehension of the conditions.

In practice in New York sufficient attention is not paid to the economics of the operating engine, and it has been a continual struggle with owners and architects to get them to spend the necessary amount, which, after all, is small, to provide a compound engine or a triple-expansion elevator pump. In some cases they are now beginning to do it, and the difference is plain and marked. It is simply expressed in terms of economy between the ordinary single-cylinder engine using 45 to 50 lbs. of steam per indicated horse-power, as they very often do, and a compound non-condensing engine affording the same for 24 lbs. per indicated horse-power per hour. But the proper load to apply to such machinery is naturally their properly designed load, and not an over-load. If the plant is designed with a suitable division and size of units, it may be that, under all conditions, those units are being used which at their economic load will take up the then requirements.

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Mr. Bolton. In water supply and sewage disposal, the elements which the author brings forward should be redistributed in the order of statement. The use of water in refrigerating apparatus is a matter which may be of interest to engineers, and to water department engineers, especially. Some of the refrigerating plants, which are now being used in large numbers in New York City, are using Croton water, direct from the mains, and are discharging it, heated to about 90°, directly into the sewers. An ice machine will use as much as 30 galls. of water per minute, per ton of refrigerating capacity. Some of these buildings have plants of 20 to 50 tons' capacity, operating 16 hours a day. This is a wrong use of city water. New York City has gone to millions of dollars of expense to provide water for its absolute needs, and cannot provide water at the price of $1.00 per 1 000 cu. ft. for profitable private refrigerating purposes or for making ice at a private profit. Of course, at present it is a commercial transaction on the part of the owner. He buys from the city water which other people are short of, and for which they are willing to pay the same price for necessities, and he uses it for making ice at a less price than he can buy ice outside. But, even as the system goes, more use should be made of this water. There is being wasted in this way a very large quantity of warm water which might just as well be used for the warm-water hand-basin system. In a building using 77 000 galls. a day, the speaker arranged that the water from the refrigerating plant, after passing over the condenser, ran into a storage tank, and after being pumped into a pressure drum, passed through the usual heating coils up to the general warm-water supply system of the toilet rooms, of which there are 140 in the hotel. That system has worked admirably, and has cut the water bill down very much. A still more elaborate system is to be installed in a new building with a larger plant, where there is also a swimming-tank, in which warmed water will be used. During the season when warmed water is required it is supplied by running it over the absorber or condenser, and then, after use in the bath, the surplus water is to be drawn off and utilized to supply the water-closet flushing system of the building. In that way good necessary use can be made of the water after it has done the work of refrigeration. It is really used twice over, the owner also making a better profit out of it. The city, however, as usual, gets the small end of the business.

The supply of water to boilers can also be dealt with in an economic way. The feed water may be heated up to about 100° economically by passing the cold supply coming from the city system through a set of pipe coils located in the drip drum or tank, where the vapors or drips from the engines are gathered.

The quantity of water which the author calculates to be used in these buildings is based upon square feet of office area at the rate of

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$\frac{4}{10}$ gall. per day for flushing water-closets, which is rather too much. The quantity of water used in a tall building is, of course, considerable. The Postal Telegraph Building is 15 stories high, and has six elevators. There is a club at the top of the building and a large kitchen in connection with it. The quantity of water used in this building is excessive. At first, when the building was opened, the quantity used per month was from 120 000 cu. ft. in the winter to as much as 190 000 cu. ft. in the summer. Then some faucets in the sinks in the restaurant were cut out, some connections in the barber shop were closed, and thus the consumption was cut down to an average of 145 000 cu. ft. per month, or about 4 800 cu. ft. a day. Mr. Bolton.

An ordinary office building with an area of about 4 000 to 5 000 sq. ft. and 15 or 16 stories high, uses about 15 000 galls. a day. One of the most wasteful elements is the self-flushing urinal. The regulations of the Building Department of New York City, which assuredly require a change, will not allow the use of a pull-chain device for flushing urinals, but require an automatic system. These are commonly left turned on, running day and night, operating every few minutes, according to the regulation applied to them, and thus wasting a very large quantity of water. That rule might very well be amended if the Water Department and the Building Department would get together, which they never seem to do.

Another waste of water is made in washing down sidewalks. The Water Department has a rule that "the use of hose for washing sidewalks, stoops, areas and house fronts is prohibited," yet any hotel structure put up in the last five years has hose plugs or sill cocks in full sight on the outside of the building, put there for that very purpose, and they are not interfered with. Another rule is that "fountains or jets in hotels, porter houses, eating saloons or other buildings are strictly prohibited." Still, in a number of hotels and department stores the fountain is a particular feature.

The pumping of water in these buildings, in New York City, is now commonly done by the direct-pressure system, and without tanks on the roof. In 1896 the speaker designed the double-pressure system of pumping, one pressure to supply the lower stories, and a higher one to supply the upper stories. For some office buildings, that has proved economical, as in the Bowling Green Building, where there are large toilet rooms on the eighth floor, and for which, and for the floors below, the lower pressure is maintained, and another pressure for all floors above, thus cutting the total pumping down to one-half. In the Ansonia Hotel, where 420 toilet rooms are to be installed, the introduction of the two-pressure system will cause a considerable reduction in the cost of pumping. In that case the mains for the eight lower floors are to be carried through the basement and deliver upward to the ninth; the upper part is to be carried to the seventeenth and de-

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Mr. Bolton.

liver downward. The drips from the bottom of the upper risers are not necessarily to be extended below, because the lowest faucet drains the pipe, and the cut-off valve is on the seventeenth floor, where the riser is shut off for repairs. The same remark applies to the warm-water system in all those buildings.

In connection with fire protection for office buildings, six or seven years ago, the Fire Department of New York City had no regulation or requirement whatever. Some owners even defied their architects and engineers to make them put in any fire protection at all in fire-proof office buildings, and others, regarding it as an ornament, spent very little money on it, and for that reason the fire appliances, generally, were inefficient and very badly designed and constructed. The speaker conferred with Hugh Bonner, late Chief of the Fire Department, and explained to him what was to be done in the Bowling Green Building. He was urged to make some sort of regulation whereby risers, extending from the street, where fire engines could make connection with them, should be put in all buildings, and an air vessel provided so that when the street engines were connected to the system the pressure would not burst the pipes if any or all of the valves were shut off. After Chief Bonner had worked on it for some time, a change of administration came, and now a regulation has been issued providing one size of risers for all buildings over a certain height. This is one of those peculiar cast-iron rules which admits of no argument. It is difficult to understand why a 6-in. riser should be necessary in a building only 120 ft. high, and yet the same be required in a building 320 ft. high. To make the owner of the small building put up such a large pipe is an unnecessary infliction, and a pipe of that size is inadequate for the tallest structures.

The system of putting out fires with water, however, is becoming antiquated. There is no doubt that more damage is often done by the Fire Department with water than is caused by the fire itself. The flooding of a building containing valuable decorations and valuable material of all sorts is a very antiquated method. Office buildings might be provided with a chemical extinguishing system, with a nozzle and valve in every room, or on every floor; then, if a fire starts in any room, close the room, or the floor, and turn on the chemical extinguisher, or even fill the room with steam, as is now done on shipboard. Such a system of fighting fire in office buildings would be far ahead of our present methods. In the German-American Building there is a restaurant in the basement, and the janitor has apartments on the roof. On a Sunday afternoon a fire started in a linen closet in the restaurant, while the janitor was asleep upstairs. The Fire Department broke in and, in putting out the fire in this closet, did $3 000 worth of damage with their axes, and the engineer and janitor did not know anything about it, yet they had complete fire-extinguishing apparatus in the building.

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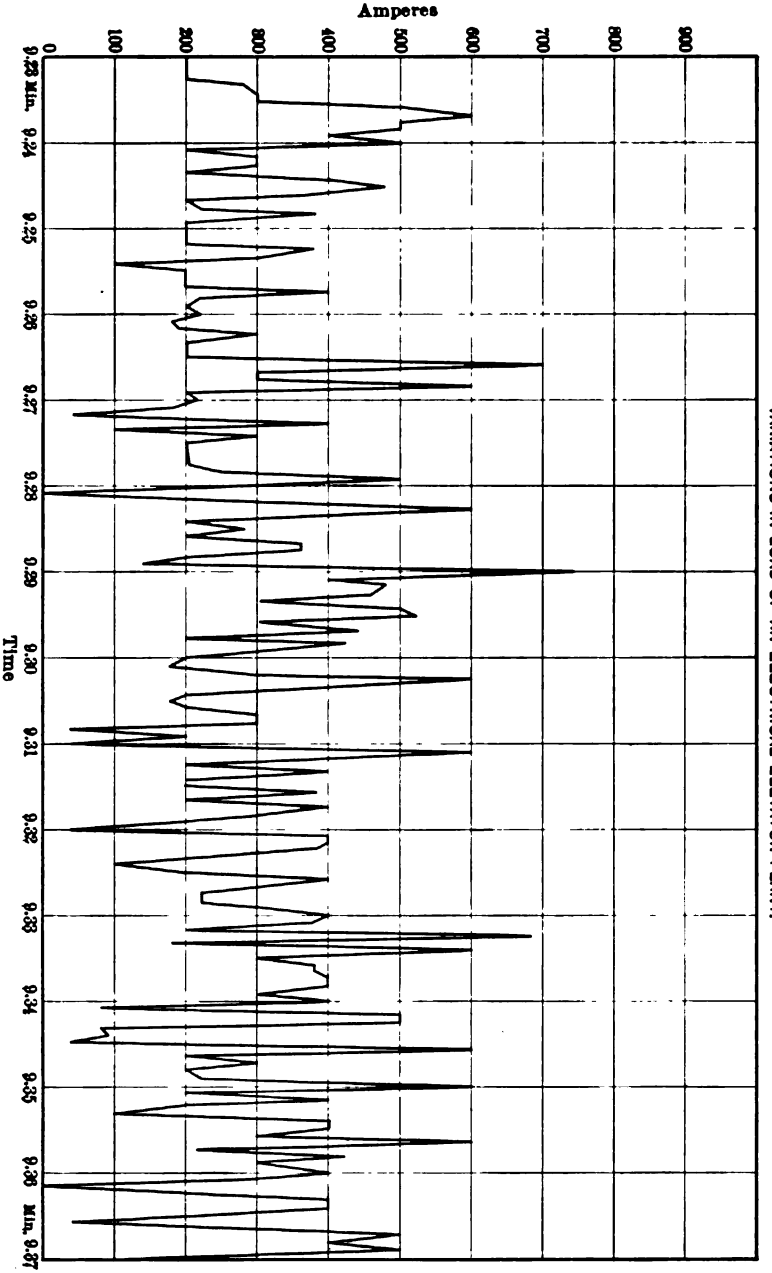
Mr. Bolton.



VARIATIONS IN LOAD OF AN ELECTRICAL ELEVATOR PLANT.

Fig. 1.

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Mr. Bolton. The author has given a very considerable dissertation on the subject of elevators. About three years ago the speaker wrote a paper,* dealing with that subject, and, without having the pleasure of knowing the author of this paper at all, arrived at much the same conclusion, that some method of proportioning elevator service was needed. He suggested the basis of 16 000 sq. ft. of rentable area for each elevator, but with the limitation that the building should not exceed 16 stories. In studying the question of elevator service it must be considered that the further away from the street the upper floors are, the worse the service they are likely to have. In a building in which all the cars are accommodation, the lower floors have cars passing them all the time, and thus get a larger number of calls than the floors further up. In attempting to provide for that, some owners and architects have divided the service as the author describes, making some cars stop at certain floors and others run express for certain distances. The St. Paul Building, for instance, has 25 stories, and has six cars; two serve eight floors; two serve from the eighth to the sixteenth, not stopping below the eighth; two serve from the sixteenth (not stopping below that point) to the top floor. This is equivalent to three 8-story buildings standing one on top of the other, each having only two cars. Every part of that building is very badly served, as offices in an 8-story building of that area, and with only two elevators, would be poor property at the present time. That shows that the building is not designed with the proper elevator service.

The Surety Building, which is 21 stories high, has only five cars. Two are express, and do not stop below the fourteenth floor, leaving a 14-story building to be served by only the remaining three cars, aggravated by the fact that those cars have also to run as accommodation cars 8 stories above. That building is also very inadequately provided, and, in order to overcome the difficulty, which the architect evidently knew would be experienced, he made large cars, to be operated at very high speed. Those are both features which are undesirable. In the first place, if very large cars are put in, a very unnecessary amount of dead load is carried at all times, and very large operating machinery has to be put in to operate a small average load. In the second place, if the speed of the cars is above a certain rate the public will object to it. The speed of a number of cars which have been running at 700 ft. per minute has had to be reduced to 600 ft. per minute because the public objected to it. A person who rides on the same car frequently, gets used to almost any speed, but a great many people, particularly the women who are largely employed in these office buildings, heartily object to any speed which exceeds about 450 ft. per minute, especially on the down trip.

In endeavoring to compare the advantages of the hydraulic and

* *Minutes of Proceedings*, Institution of Civil Engineers, Vol. cxliii, page 218.

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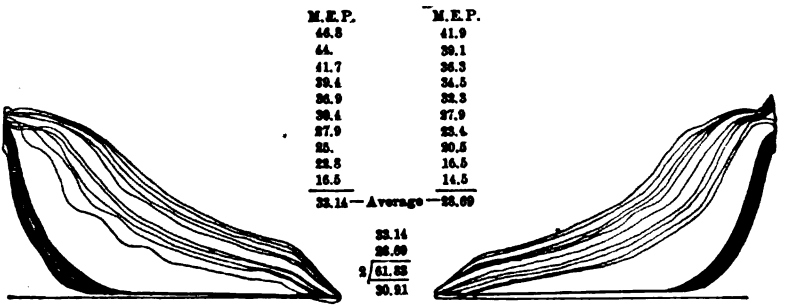
electric operation of elevators, the author is very wide of the mark. Mr. Bolton.
He asserts that:

"The hydraulic elevator, unless complicated with a differential water supply, uses the same amount of power for light as for heavy loads. The hydraulic ram elevator is limited in speed and range. The electric elevator uses power in proportion to the load, and is more economical than the hydraulic machines."

That is not correct in either statement. Theoretically, the electrical elevator might use power in proportion to the load, but the load of an elevator is a series of jumps, and is illustrated in the diagram, Fig. 1, showing the variations, taken at 5-second intervals, of a 3-elevator plant. Fig. 2 is a set of diagrams, taken at 1-minute intervals, from an engine driving such a plant, and shows how the power varies, and what a series of blows upon the generator is caused by the sudden



INDICATOR DIAGRAM
TAKEN DURING TEST AT LORDS COURT BUILDING; 1.35 P.M. JULY 30TH 1896.
AMES ENGINE NO. 14737—16″X 14″—246½ REVOLUTIONS. SCALE OF SPRING = 50.
TEN CARDS AT ONE MINUTE INTERVALS.
I.H.P. BY LARGEST CARD = 154.
I.H.P. BY SMALLEST CARD = 54.
I.H.P. AVERAGE = 108.

FIG. 2.

changes. Only those who have stood by such engines and have seen the strains to which they are subjected can appreciate how difficult it is to have machinery strong enough to stand such work. A diagram taken at a time when four cars started together figured out over 250 H.-P. on a 17 x 16-in. single-cylinder engine, and, of course, under those circumstances, it would be impossible to get any real economy out of the engine because of the varying load. The net result is that the electrical elevator system, driven by an engine and generator direct, is not and cannot be economical. It is, in point of fact, the generation and distribution of power over a short distance to a motor which is constantly varying its load, and there is no storage between it and the motor. If a storage battery is provided, to overcome the difficulties of variation, as in the case of the Commercial Cable Build-

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Mr. Bolton. ing, or the R. G. Dun Building, it is only obtained at the cost of the storage battery. In the R. G. Dun Building the maximum demand, which ran up to 1 600 amperes, was reduced to a level average of 320 to 340 amperes by placing a storage battery in parallel with the engines, but the storage battery cost nearly $30 000, and its depreciation must be reckoned at 7% per annum, and its capital outlay at 5%, compound interest, when any possible idea of superior economy, over a hydraulic plant of equal proportions, absolutely disappears.

Of course the battery pays its way by showing better results than without its use on the electrical plant. But it certainly would be an economy to replace the whole by a hydraulic plant of modern design. The first commercial plant of electrical elevators in the Syndicate Building was installed in 1896, and had not run for any length of time before the foregoing conclusions became clear. These remarks apply to office, hotel and store conditions; where elevators are run on schedules; where loads vary, but speeds must be maintained. For isolated elevators, such as in small apartment houses, the drum electric elevator is desirable.

The question of repairs is one that has been fought over very considerably, but when it is considered that in the ordinary high-speed elevator the life of the screws as first put in is only two years, and that the friction gear, the ball bearings, and many other details, require constant renewals, it will be seen that that also is another very expensive feature. In the case of such a building as the Dun, which is maintained at a high degree of efficiency, they can do so only by having a man standing by the elevators all the time. If such a plant is charged with $1 200 a year for extra labor, a considerable economy in coal must be shown as against a simpler system.

But the hydraulic system is not so uneconomical as represented, and neither is it so complicated, when a differential system is used. In the Bowling Green Building the duplex system of pressure has now been working successfully for 4½ years. In that plant a triple-expansion engine pumps all the water required by the entire elevator service, but only to 120 lbs. pressure, sufficient for use with the average loads; a second compound engine then draws water from the first pressure drum at 120 lbs. pressure and pumps to 210 lbs. in a second drum. The two supplies are piped over to the main valve, which is made with an extra lift, so that by pressing the lever in the car over to the last notch water at the higher pressure can be taken into the cylinder. The cylinders are then reduced in size, and with the low-pressure water are capable of carrying the car only with the average load at the proper speed. Instead of being 15½-in cylinders they are only 12½ ins. in diameter. Under ordinary circumstances, that is, seven-ninths of the time, the operator carries his average load and gets his rate of speed by the use of the low-pressure water, but when

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he has an additional load he puts his lever over to the farthest notch and takes in the high-pressure water. Mr. Bolton.

Another element, always forgotten in this connection, and interesting to a hydraulic engineer, is that, if, as stated by the author, the same quantity of water is used in a hydraulic cylinder for no load, or for a light load, as for a full load, it must not be forgotten that, with this water and the lighter loads, the car goes faster. If the water is turned into the cylinder, and the resistance of the load is very little, the car runs that much faster, and that is an improvement in the elevator service. If the load consists of only two people, and they are to be taken nearly to the top floor, the round trip can be made quickly, and in that way the car is sooner in service again. The mileage gained has to be considered as an advantage. It is not a loss, although, on the face of it, it may appear to be so.

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*Referring to the comparative merits of the hydraulic and electric elevator systems, the diagram, Fig. 7, only illustrates a fault, common to very many hydraulic installations, that of over-proportioning the pumps. There appears to be very little judgment exercised in the matter. If the pumps referred to had been smaller, their operation would have been more continuous, and, when accompanied by an increase in the storage capacity, the fluctuations in power shown by the diagrams would be reduced.

In an installation in a building on Broadway, New York, in 1897, the writer carried out these features with most satisfactory results.

The entire comparison has recently undergone a radical development in favor of the hydraulic system, by the demonstration of the efficiency of the plunger or direct-acting elevator, the water consumption in which is from 30 to 40% less for a given average duty than the geared apparatus.

The writer does not at all agree with the author's alleged incidental advantages of operation of electric elevators, especially as the attempt to operate the electric machines in conjunction with a lighting load has proved a failure time and again, and can only be effected by the addition of a storage battery.

The operation of a hydraulic system by electrically-operated pumps is questionable, and, on any large scale, has proved undesirable.

Electricity is essentially unsuited and unnecessary as a means of operating intermittent hoisting machinery, such as elevators, placed as they are in immediate proximity to the source of power. Where power has to be distributed to numerous and distant points, its advantages for transmission may be pronounced; but every element in an elevator plant run on schedule is adverse to the elementary

* Discussion continued by letter, after reading that part of Mr. Darrach's discussion on pages 45 to 69.

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Mr. Bolton.

features of electrical transmission, and the entire abandonment, by its manufacturers, of the screw-type, and their relegation of the drum-type to isolated installations, is the best evidence of the error made in endeavoring to adapt this transmission to a service so evidently and desirably adapted to the features of hydraulic transmission.

To the writer's statement that the cubical contents of an office building have no relation to the boiler capacity required therein, the author retorts with a combination of formulas for the calculation of heat losses, all of which are well known, and are utilized in one form or other by any well-informed heating engineer. The particular error of assertion, on which the writer's remarks were based, however, is not controverted. The operating services in such buildings may and often do exceed the heating capacity, and the horse-power to be installed must be settled first by reference to the requirements of those services, and then compared with the heating needs. In the formula, the element, N, is scarcely appreciable in New York City, as the interior air of most ordinary office buildings is stagnant, and the cubical contents, therefore, are not an element in either direction.

The "double-supply" system of steam piping, in which the author appears to have a paternal interest, presents no novel features, and, in point of desirable arrangement, is inferior to a system of supply to the middle of every riser. For convenience, the system of overhead supply is of superior advantage, as the riser pipes then are reduced in size through the lower stories, where they present the greatest architectural difficulty. Therefore, if an overhead distributing system can be arranged in an attic or top story, there is a disadvantage in adding thereto an unnecessary and therefore undesirable lower system of supply lines in a cellar or basement. On the downward-supply system, with a vacuum return, the risers need be of no larger size than those the author appropriates to his own method.

The author's suggestions for discounting the disadvantages of the electric light are not practical. The ordinary incandescent lamp affords its maximum lighting capacity at right angles to the axis of the filament, and not vertically. Therefore, when placed at a proper height, the rays are diffused sideways.

To turn the lamp upside down and then attempt vertical reflection is to invite that failure which the ignorance of architects is constantly repeating.

In the Shelby lamp the carbon is arranged to emit its full candle power from the end of the bulb, and in all positions where a direct light is required, it forms a most useful improvement.

If, as the author suggests, "for general illumination," the lamp be placed bulb end upward, there is an initial loss of 40% of its illuminating effect on the floor. The further screening with a frosted globe would cut the side rays down at least 50%, and the final reflected effect

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from a whitened ceiling affords in many such instances a bare 10% of the light produced. It is earnestly to be hoped that no engineer or architect will be misled, by the author's panacea, into regarding such methods as the limitation of scientific arrangement and utilization of the electric light. Mr. Bolton.

CHARLES G. DARRACH, M. Am. Soc. C. E. (by letter).—In reference to Mr. Bolton's very interesting discussion, relative to the various necessities noted on page 2, the writer cannot see that one is more important than another. An office building without light and water supply would be as absurd as one without heat, elevators or sewage disposal. A certain amount of artificial ventilation is also necessary, and, whatever may be said of the ventilation of the various offices, provision must be made for the artificial ventilation of the engine and boiler-room and toilet-rooms.* Mr. Darrach.

In Philadelphia, the ground floor and part of the basement are generally used by banks, trust companies, and kindred institutions, in which there are many clerks. For such conditions, warm-air heating and ventilation is generally demanded. Mr. Bolton states that this is not the usual practice in New York City and surrounding districts.

It is needless to state that to determine absolutely the boiler installation required, the only method is that suggested by Mr. Bolton, but the writer must take issue with the statement on page 29 which states:

"The speaker regrets that he has some contradictions to apply to some of the figures presented in the paper, but, as regards New York practice, they certainly should not go forth as being definite.

"To begin with, the author indicates the size of the boiler installation, basing it upon the cubical contents of the building, a most

* To ventilate the boiler-room, an adaptation of the usual method adopted in regulating the steam pressure on small house-heating plants can be advantageously used. Instead of operating the damper in the smoke flue, to choke the draft by reducing the area of the flue, the writer provides a check-damper in the flue, admitting air to the chimney from the upper part of the boiler-room, and constructs a flue from the ash-pit to the upper part of the boiler-room, also controlled by a damper; the damper admitting air under the fire and the damper admitting air to the chimney are controlled by the steam pressure in the boiler, the ash-pit damper being closed as the chimney damper is opened.

This method insures a continuous flow of air from the upper part of the boiler-room, producing an excellent ventilation, reducing the likelihood of the explosion of unconsumed gases in the flue, as well as admitting warmed air for combustion.

In Philadelphia, vaults under the side-walks are frequently used as a part of the engine and boiler-room, and these rooms are ventilated to the street through openings in the pavement. This practice is very objectionable, and should be prohibited by law.

The engine-room can be ventilated either by a shaft running to the roof, or by a system of plenum and exhaust fans.

The proper ventilation of the engine and boiler-room is often neglected, and there is no necessity for requiring the machinery and employees to be subjected to a temperature as high as 120° to 130°, as is sometimes the case.

In one instance the writer was obliged to construct a very tortuous chimney (limited in height) and a long horizontal flue, having in its course some seven right angles; the exhaust fan in the engine-room was connected to a flue arranged to deliver the air either to the ash-pits of the boilers or directly to the chimney. The flue serving each boiler was provided with a pair of dampers operated by the steam pressure, so that the air either passed through the furnace or directly to the chimney, as needed; by this method the engine and boiler-rooms were thoroughly ventilated and kept cool, and the economy and capacity of the boilers increased.

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Mr. Darrach. misleading element, as the cubical contents have no relation to the horse-power required."

The writer thinks that this criticism is incorrect. The boiler-power required for the power plant, including lighting, elevator service, pumps, fans, etc., as well as heat required for change of air in ventilation, depends directly upon the cubical contents, and the only variation is the proportion of glass surface and the exposure.

For the amount of direct radiator surface required, the writer uses the formulas:

$$R = h \frac{t - t_1}{(T - t)a};$$

$$h = (k\,G + K\,W + 0.018\,N\,C)\,E;$$

in which R = square feet of prime radiator surface;
G = " " " exposed glass "
W = " " " " wall "
C = cubic feet of space to be heated;
N = number of times air is to be changed per hour;
k = B. T. U. transmitted per difference of 1° per hour per square foot of glass surface;
*K = B. T. U. " " " of 1° per hour per square foot of wall surface;
a = B. T. U. " " " of 1° per hour per square foot of radiator surface;
h = total heat units transmitted per difference of 1° per hour;
E = coefficient of exposure;
t = required temperature of room;
t_1 = temperature of outside atmosphere;
T = " " steam in radiator;

If $$P = \frac{Ra}{h} = \frac{t - t_1}{T - t},$$

then $$t = \frac{PT + t_1}{P + 1}$$

and the result is a formula by which the heating power of a system can be equated at the temperature of the outside atmosphere, other than that for which the system was designed. The observations should be made without the modification due to wind storms.

The writer has verified this formula by observations with the outside atmosphere from 7 to 30° Fahr.

The minimum temperature of — 6° at New York and Philadelphia is from the records of the United States Signal Service Bureau. The

*$K = \frac{5.236}{\Theta + 3.7}$, for good brick or stone walls; Θ = thickness of wall, in inches.

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writer has observed temperatures below zero; he states, on page 3, "the temperature often falls to + 10°, and sometimes below zero. Wind storms of 25 to 45 miles per hour are not infrequent." Mr. Bolton suggests 60 miles per hour, which, from the writer's observation, is as infrequent an occurrence in this vicinity as a zero temperature. Fortunately, these two conditions rarely, if ever, obtain at the same time, for in very cold weather the wind velocities are generally low. Mr. Darrach.

In computing the radiator surface, the writer figures from 70° Fahr. to zero, for heat losses from walls, windows, etc., allowing one or more changes of air in the room per hour, depending upon circumstances, and adds 10 to 15% on the north and west, and for severe east exposures.

From the writer's experience in Philadelphia, the result of this method indicates the proportions per cubic foot noted. Mr. Bolton states, for approximation, 1 H.-P. for 4 000 cu. ft. From the writer's figures, this approximation would be 1 H.-P. for 7 000 to 5 000 cu. ft., with additional boiler units in reserve. Of course, in more severe climates these figures would be increased. Mr. Bolton does not give the cubic contents of the Hudson Building or the German-American Building, nor the ratio between the cubic contents and the boiler power. As space for the installation of the boilers and machinery is often limited, advantage might be taken of the maximum evaporation of the boiler plant for the few days during the heating season when the weather is most severe. These spells of very severe weather last generally only a few hours, sometimes for a day or two.

The heating diagram (Fig. 3) was made from data taken at the office building of the United Gas Improvement Company, at the northwest corner of Broad and Arch Streets, Philadelphia.

This building has 12 stories and an attic, the ground plan has an area of 8 814 sq. ft., and, above the first story, an area of 7 386 sq. ft., with a width of 57 ft. Above the sidewalk, and within the exterior lines, the building contains 1 320 000 cu. ft.

There are, in all, 650 windows, and a large skylight, having in all 16 000 sq. ft. of glass, or about 25% of the exterior exposure. These windows are furnished with sash fasteners and "Golden" metallic weather strips.

The first story is heated with warm air, supplied by fans. The stair hall, elevator shaft, and the building above the first floor, are heated by direct radiators under the windows. Exhaust ventilation is provided for the entire building, with fans in the basement for the first story and below, and in the attic for the rooms above the first story.

There are 2 800 sq. ft. of radiator coils for the indirect, and 12 700 sq. ft. of direct, radiation, making an equivalent of about 19 000 sq. ft. of direct radiation.

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Mr. Darrach. As power for elevators, lighting, fans, pumps, etc., is generated by gas engines, and the heating is entirely independent of the power, an excellent opportunity is presented for obtaining exact data as to the heating requirements.

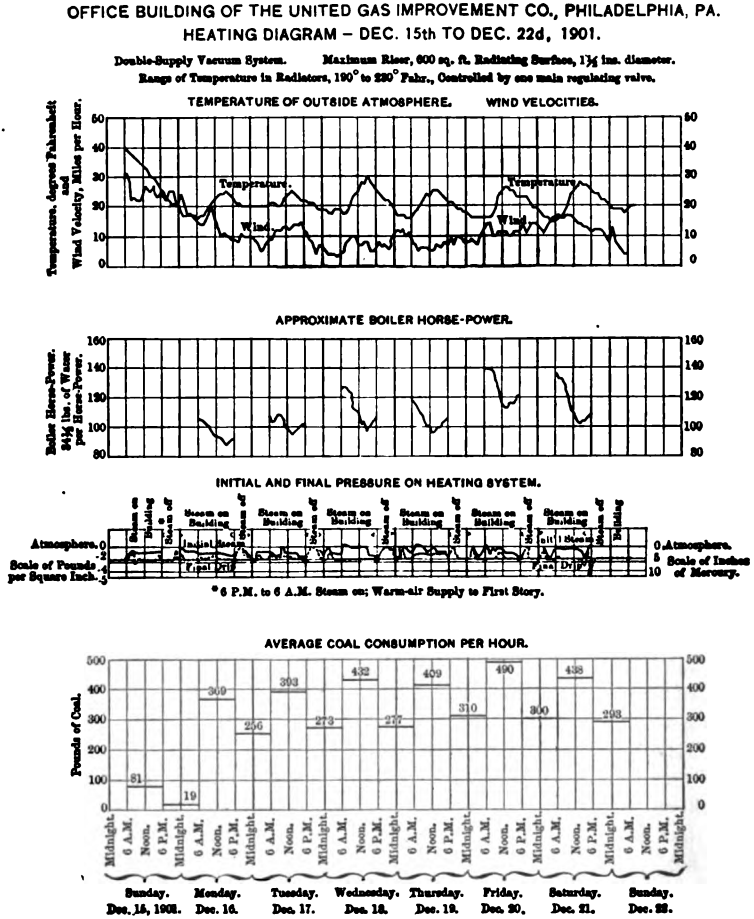

FIG. 3.

Upon the diagram (Fig. 3) profiles made from hourly observations are shown:

1. The temperature of the outside atmosphere, in degrees, Fahrenheit.

2. The wind velocity, in miles per hour.

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3. The approximate boiler horse-power (6 A. M. to 6 P. M.), calcu- Mr. Darrach.
lated from the coal burned, rating 9 lbs. of water evaporated per pound of coal, and 34½ lbs. of water per horse-power-hour.

4. The pressures (below the atmosphere) on the steam main in the boiler-room (initial), and on the drip-pipe (final).

5. The average number of pounds of coal burned per hour, in 12-hour periods.

From these data it will be found that 185 boiler horse-power would be required to heat the building at zero Fahrenheit, or, say, 7 000 cu. ft. per boiler horse-power.

The steam supply to the radiators is distributed on the double-supply system. The largest steam riser is 1½ ins. in diameter, and serves 600 sq. ft. of radiation. A range of temperature of steam in the radiators from 190 to 230° has been obtained by the operation of one valve, regulating the temperature in the radiators to suit varying conditions of weather.

The usual practice in calculating radiating surface is to figure the heat losses from the outside surface, with an allowance for cubic contents, and then add a percentage to cover additional losses due to severe exposure, without, as Mr. Bolton says, considering the height of the building.

The rule given on page 33 is not definite. By it the calculated radiating surface is equated for the various floors by "using an addition for the height of each floor by adding an amount proportioned on the square root of the height from the street, and in that way a small amount of surface was added for each floor." What does this mean?

If a story height is used as the unit of height above the street, the coefficient for the sixteenth story would be 4. It would be absurd to say that the sixteenth story would require four times the amount of radiation required by the first, and if the usual 10 or 15% allowance is multiplied by the square root of the height the addition would be approximately 50%, which is also absurd.

If it has been found that the upper stories of the building require a more liberal allowance of radiating surface than the lower stories, may not the reason have been that the steam supplied from the basement (which would require for the sixteenth story a riser approximately 200 ft. long) had become attenuated by friction, condensation, and extra demands on the lower stories?

A greater amount of radiating surface would be required, to give expected results, if the temperature of the steam was lower.

The writer has found, when using low-pressure steam with the supply from above only, that the lower stories suffered.

These facts go to show the advantage of the double supply to the risers, *i. e.*, a main feeder in the attic as well as one in the basement. These feeders have approximately half the area of the ordinary feeder,

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Mr. Darrach. which supplies at one end only, and, generally, can be installed at about the same, sometimes at much less, cost.

Whatever the rule may be, it appears to be a refinement beyond the possibility of general application, for instance, in the case of the "Lord's Court Building," noted on page 31.

When a wind storm strikes a tall building the wind is deflected down to the pavement, sweeping the entire structure, and, in locations where tall buildings are on the corners of intersecting streets, the velocity of gusts at the pavement may be greater than the storm at higher altitudes. The writer has frequently been obliged to seek a friendly lamp-post to prevent being blown away.

The main reliance must be on tight windows, etc. The instance stated on page 3 is an example. The rooms on the seventh floor could not be heated to over 62°, with the outside temperature 17° and the wind at 25 miles per hour, even with the initial steam at 218°; but, after the windows were weather-stripped and the leaks were stopped, the rooms on the twelfth floor were heated to 70°, with the outside temperature at 17°, the wind at 37 miles per hour, and the steam at 212 degrees. Another record shows rooms on the twelfth floor at 70°, outside temperature 25°, wind 25 miles, and initial steam temperature 208°; and another record, rooms on the twelfth floor at 70°, outside temperature 12°, wind 20 miles, and initial steam temperature 206 degrees.

Too much stress cannot be laid upon the necessity of tight window frames and sash, for without this provision all calculations for the amount of heating surface required will go astray. As stated by Mr. Bolton, the first winter is the one in which the building is in the worst condition for heating. One reason is that the large quantity of water used in the walls and cement floors must be dried out, especially if the building has been finished late in the fall.

The setting and design of window frames to prevent leakage, without relying upon the questionable expedient of filling joints with plaster, should, without doubt, be impressed upon both builder and architect. In the writer's experience, it has been necessary to caulk these joints with oakum, and then not get a good job. The question is a difficult one, for, even if the frames are made of seasoned lumber, they are apt to shrink after the building is heated.

Mr. Bolton's remarks on the use of direct-indirect radiators in offices have a great deal of force, for, no matter how well the apparatus may be designed and constructed, the tenant must continually adjust the inlet damper to suit the conditions of the prevailing wind. If the inlet damper is allowed to remain open at night, and the steam supply is reduced (which is generally the case), it is probable that the radiator will freeze during a cold wind storm, unless a vacuum system is used.

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Mr. Darrach.

Referring to Mr. Bolton's remarks on chimneys, the architect probably was not aware that the area of the chimney was not the only element required for his information. The difference in value of different sections of the same area would never enter into his head, nor would he know that connecting the two flues at the top would practically destroy the value of the chimney.

To harmonize the requirements of the engineer in designing the mechanical plant and the architect in designing the building, so that the best available results may be obtained, requires more patience and ingenuity than those without experience can imagine. The engineer naturally desires the building to be designed and constructed to accommodate the mechanical plant in the best manner, whereas the owner and the architect generally care for none of these things; therefore, the engineer is given the smallest available space, without regard to the necessities of the case. A prominent example of this condition of affairs may be seen in the building of the Land Title Trust Company, in Philadelphia, where double the space allowed for the engine and boiler-room would not be too much. It is also often difficult to have provision made so that the machinery, boilers, stack, etc., may be repaired or renewed; the prevailing idea being, "once in, always in."

The writer would design the chimney for the most severe conditions, but, if limited either in space or cash, he would feel safe in driving the boilers to say 30% above their rating, during periods of maximum demand.

With the high chimneys available in most office buildings, water-tube boilers with 11 sq. ft. of heating surface per horse-power can readily be run from 30 to 40% above their rating.

Referring to the Webster Vacuum System, there has been a great deal of complaint on account of the thermostatic valves choking. The writer finds that, if the radiators are pickled and the moulding sand and scale are removed before installation, little difficulty from this cause may be expected.

Another source of complaint is that, after continued use, the composition plugs in the thermostatic valves lose their temper. This difficulty is easily remedied by renewing the plugs, a matter of little expense or difficulty, and which should not be considered in comparison with the advantages of the system.

To obtain good results, and prevent short circuits, with the Webster system, it may be advisable to run independent drip-mains from certain portions of the system directly to the pumps, and not connect the entire drainage system into one main. With the Vacuum System and an equable distribution, the heating system becomes the surface condenser, and, excepting in cases of very severe weather, when a steam pressure above the atmosphere may be required, prevents all back pressure on the engines, and generally produces a slight vacuum.

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Mr. Darrach. The vacuum pump required to remove air and condensation from the radiators requires only a trifling expenditure of power. The writer found that the power required to operate an electrically driven pump on the Vacuum System, serving 15 000 sq. ft. of radiating surface, did not exceed 3½ H.-P.

Another advantage of the Vacuum System is that, in concealing the pipes, there is no danger from leakage, as the system is generally operated at or under the atmospheric pressure, and if there is leakage, the tendency is for air to leak into the pipes, rather than for water and steam to escape from them.

To illustrate the operation of the double-supply distribution combined with the Webster Vacuum System, the writer submits the diagrams, Figs. 4 to 7, showing the pressures on the steam supply and drip-pipes from Friday, January 18th, 1901, at 9 A. M., to Sunday, January 20th, at the same hour. On the diagrams showing the steam pressures on the supply pipes the hourly outside temperature and wind velocities have been plotted, and show how the wind velocities reduced as the outside temperatures lowered. The temperature in the building was maintained at 70° during office hours, and lowered slightly during the night, after steam was shut off. On January 18th the initial steam pressure on the supply pipes varied from 1 to 2 lbs. below the atmosphere from 9 A. M. until 7.30 P. M., when all the steam was shut off and the vacuum pump was stopped. The pressure was then reduced by condensation to 6 lbs. below the atmosphere. Leakage of air into the system produced an equilibrium at 1.30 A. M., January 19th, when the pump was started.

Steam was turned on the building at 2.30 A. M., and atmospheric pressure was maintained until 7 A. M., when it was reduced to 1 lb. below the atmosphere until 9.45 A. M., and varied from 1 lb. below to atmospheric pressure until 6.45 P. M., when steam was shut off, and the pressure was reduced by condensation to 6½ lbs. below the atmosphere. Leakage did not fill the system with air until after midnight. The vacuum on the drainage system produced by the pumps varied from 1 to 2½ lbs. below the atmosphere.

In the heating system from which these diagrams were taken, the steam risers were 1½ ins. in diameter, and 170 ft. long; each served about 600 sq. ft. of radiation.

The initial steam pressure on the system may readily be raised to from 2 to 3 lbs. above the atmosphere during the most severe conditions, producing a correspondingly higher temperature in the radiators.

Referring to Mr. Bolton's remarks on the wide range of temperatures to be obtained with the Webster Vacuum System, in which he states that the temperatures in the radiators "can be regulated from 90° up to full temperature of steam," the writer has never been able

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Mr. Darrach.

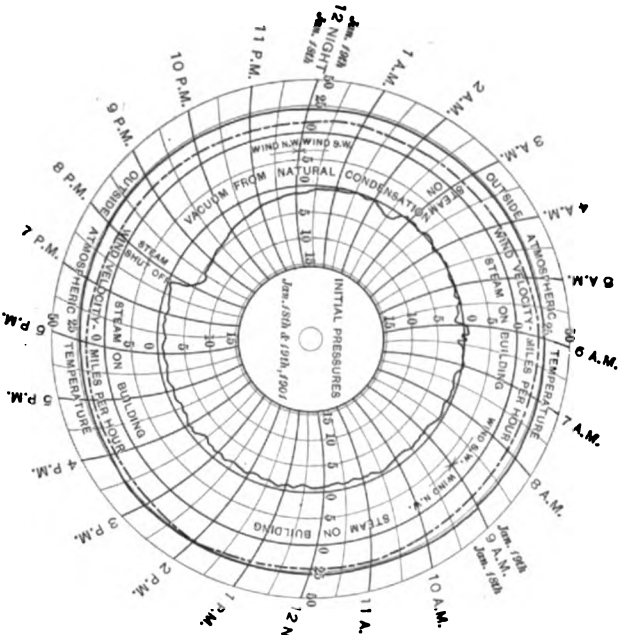

Fig. 4.

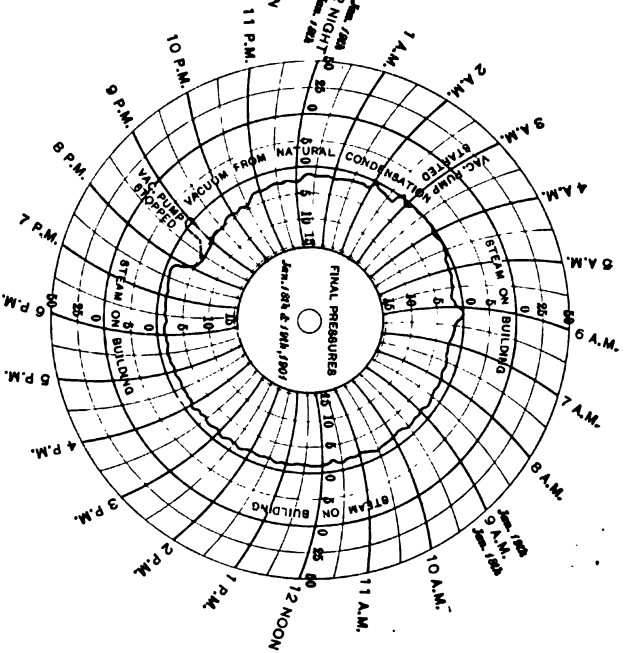

Fig. 5.

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Mr. Darrach. to accomplish these results, and he thinks it would be difficult to serve 100 sq. ft. of radiating surface with a ¼-in. pipe and get a temperature of 212° in the radiator without a high initial pressure in the risers.

With a ¼-in. pipe, serving 100 sq. ft. of radiating surface, the ratio of pipe area to radiating surface is only 0.003 sq. in. per square foot. In Mr. Bolton's opinion, a 2-in. riser, supplied with steam at both ends, serving 1 200 sq. ft. of radiating surface in a 24-story building, is too small; yet the ratio of steam pipe area to the radiating surface for such a pipe would be 0.0056 sq. in. to the square foot of surface, only at the first and twenty-fourth stories. The riser being of the same diameter for the entire height, the ratio at the middle of the building increases to 0.067 sq. in. of pipe area to the square foot of radiating surface, so that the steam velocities to the most remote radiators are reduced.

To illustrate the comparative dimensions of supply and drip-pipes, as used by the different systems, the diagram (Fig. 8) is submitted. Curve No. 3 shows the diameter of the risers for the corresponding radiator surface, if they are provided with a double supply, and the system is drained by gravity, or by the Webster Vacuum System, supplied either from top or bottom.

Curve No. 4 shows the diameters of the pipes in the Webster Vacuum System supplied from both ends.

The writer has found that the size of the drip-pipes noted in Curve No. 6 is sufficient for the risers on the gravity two-pipe system.

In tall buildings it is not considered good practice to make the least dimension of the riser less than 2 ins. diameter, increasing toward the source of supply; say, in a 24-story building, the riser serving 1 200 ft. of radiation, the largest diameter of the pipe on a one-pipe system would be not less than 3½ ins., whereas, with a double-supply gravity system, it would be 2½ ins., and with a double-supply vacuum system the diameter would be less than 2 ins.

In serving the risers from both top and bottom, the building is practically divided into two buildings, one served from above, the other from below, with the advantage of equating the variable steam demands from either the upper or lower supply. The double-supply system has the advantage of giving more equable distribution, and, as the pipes are smaller, there is less trouble in concealing them than if the entire building was supplied either from the top or the bottom.

The one-pipe gravity system, supplied either from the top or the bottom, requires pipes of maximum size, and in tall buildings they are often difficult to conceal, without taking into consideration the question of expansion.

Mr. Bolton states that it is difficult to have architects provide an attic, and virtually says that in nine out of every ten modern build-

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Mr. Darrach.

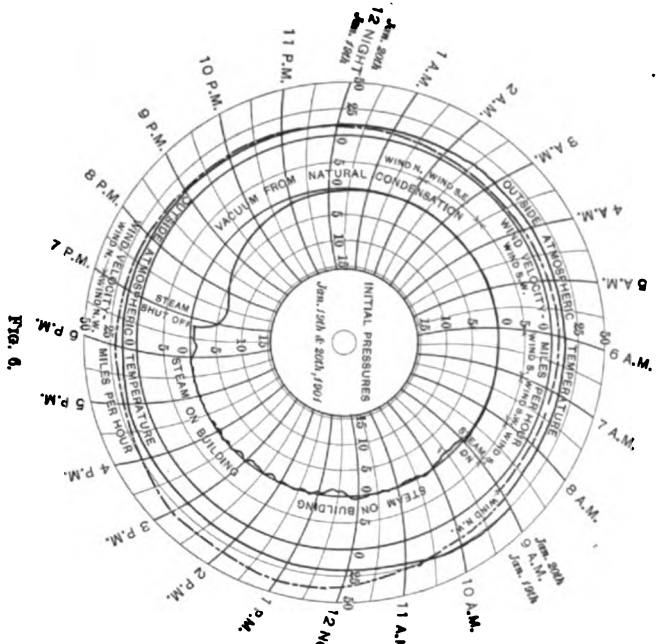

Fig. 6.

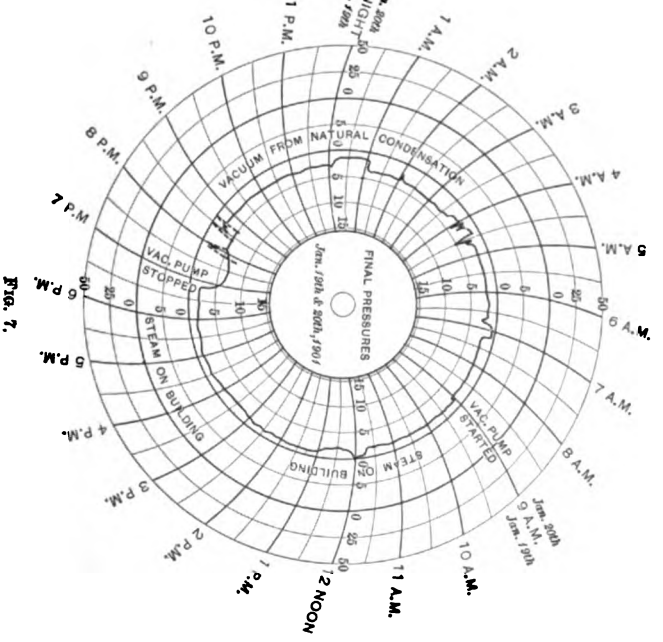

Fig. 7.

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Mr. Darrach. ings no attics are provided. This is not so in Philadelphia. In the writer's opinion, an attic is advantageous, as it may, at a less cost, prevent the construction of a sub-basement. Its construction is comparatively inexpensive, and, besides its other uses, it serves an excellent purpose in protecting the upper story from the heat in summer and the cold in winter.

In the writer's opinion, it is always better to supply the steam-heating risers from above. If there is an attic, this is easily accomplished, and relieves the "congested basement." The attic also provides a location for water-supply tanks, ventilating fans (if they are used), dressing rooms for help, and a space for storage. If the basement is used for safe deposit vaults or other purposes, and the space for engines and boilers is limited, some of the apparatus may be transferred to the attic; for instance, pressure tanks for hydraulic elevator service, cooling tanks for the refrigerating apparatus, and, if hard pressed, the electric storage battery.

On page 7 attention is called to the advantages in the use of electric lights, but, as generally used, they have some serious disadvantages. The glare from the highly illuminated filament produces a strain upon the eye which should be prevented, and some method should always be adopted to prevent this, either by the use of shades, frosted lamps or frosted globes.

A bare incandescent lamp should no more be tolerated than a bare arc light. It will be found that if the filament is obscured no serious loss results, nor will there be any necessity for an increased number of lamps. Some eight years ago, in designing the lighting of the train-shed of the Reading Terminal Station, at Philadelphia, the writer used full frosted globes on all the arc lights, and had apparently a much better illumination than was obtained in the Pennsylvania Station, where plain glass globes were used. The reason is very simple. The glare of the bare arc or the bare filament is so great that Nature closes the iris; whereas, when the light is diffused, the iris opens and more light is admitted, without as great a strain upon the retina. Disregarding this fact, most electrical engineers prescribe, as an ultimatum, that the incandescent lamp must be placed bulb end down. For general illumination, the lamp should be placed bulb end up, screened with a frosted globe, and the light reflected from the ceiling.

Some years ago, the writer had occasion to light the chancel of a church. Incandescent lamps were placed in the rear, around the chancel arch, with reflectors converging all the rays of the various lamps upon a white Parian marble altar. The diffusion of light was so perfect that the priest, standing in front of the altar, did not cast an appreciable shadow.

Mr. Bolton's experience with the 220-volt system is certainly very unsatisfactory; the writer would imagine that he would have such ex-

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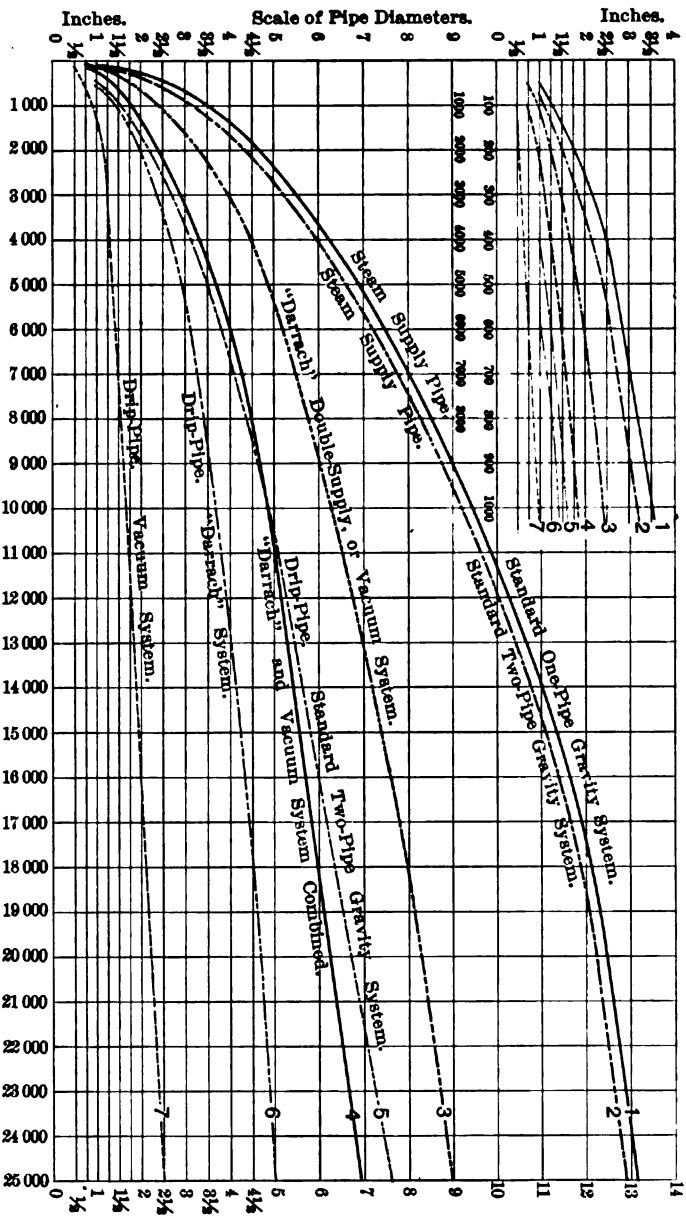
Mr. Darrach.



COMPARATIVE DIAGRAM OF STEAM HEATING AND DRIP-PIPES.

Fig. 8.

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Mr. Darrach. perience in a hotel. The trouble he mentions with reference to the distribution boxes has not come within the writer's experience, but, from what has been stated, the further adoption of this voltage should be carefully considered.

The many advantages to be gained in using this voltage should not condemn it because first attempts may show defects of installation. In the writer's practice, and from inquiry among those using this voltage in Philadelphia office buildings, the troubles complained of have not been experienced.

The higher voltage requires better installation and greater care in construction, and the fuses in the junction boxes should be arranged so that if one fuse should burn out, it will not burn out the others.

The lamps for the 220-volt current have been much improved, and no doubt will be improved further, as the 110-volt lamps have been.

The experience in Philadelphia with the practical use of this voltage has been so successful* that its further adoption is warranted, and, in the writer's opinion, "the millennium" should not be regarded as impossible because of first failure.

In the discussion, page 34, the statement that there is a word omitted on page 8, is correct. The sentence should read:

"The writer has found that the average maximum amount of current used will vary from 50 to 60% of the total installation, including the engine and boiler-room lighting."

The writer can hardly agree with the statement that, in the event of a sudden thunder-storm on a summer afternoon, every light in the building would be turned on. This would mean that every office in the building was rented; that every man in the building was at his desk; that the lamps in every closet or safe, all hall lamps and administration circuits were lighted, and that all fans and other motors were in operation. In the writer's experience, this load has never exceeded 75% of the total installation, although he does not wish to state that it is not possible for a greater load to be suddenly turned on. The total load is hardly possible.

The average load and the maximum working load, compared with the total installation, of course, will vary with every building. In buildings where there are many interior offices, or where the offices are very deep, these loads are increased, and taking care of the thunder-storm load is made more difficult.

The writer has suggested a 3-unit plant, each unit of equal capacity. This he considers a good general rule, although there may be conditions which would modify it.

Some fifteen years ago, the writer installed a 200-H.-P. plant, providing two 25% machines, and one 50%, with an additional 50% in reserve. On one occasion, one of the 25% units was jogging along

* The writer has records of 1 500 hours, as the life of a 220-volt lamp.

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comfortably, when a sudden thunder-storm overloaded the machine before the assistant in charge could throw on the second machine; in fact, he did not know what was the matter. As soon as the main fuse was blown (for it was before the days of circuit breakers) he started the second 25% unit, which promptly burnt out its fuse. He then started the 50% unit, which followed suit. By this time, the cause of the difficulty was discovered, and the two 50% units were put in commission. Mr. Darrach.

One reason for very heavy thunder-storm loads is the arrangement of switches controlling the electroliers; for instance, where the offices are arranged in suites, and all the bracket lights are put on one switch, all the pendants on another, and the two switches are placed close together near the entrance door, so that the tenant can, by two movements, light all the lamps in the suite. The writer is of the opinion that the main switch on the bracket lamps should not be installed, and that each pendant should have an independent switch.

On page 8, the writer states:

"A high-speed, single-cylinder, steam engine and electric generator, properly rated, will give economical results from 50% under-load to 50% over-load. A high-speed, compound, non-condensing engine will give economical results at half-load. If two units are provided, each rated for one-half the maximum, a 25% load can be operated without serious loss.

"Three units, each rated for one-half the maximum load, provides ample power—with one unit in reserve." * * *

It is advantageous, with a variable load, to run a single-cylinder engine above a rating at its most economical point of operation; the engine should be constructed to carry such a load without danger of break-down. It will be conceded that the steam consumption at 50% over-load generally will not exceed the most economical load by more than 10 per cent.

In one instance, the writer installed two single-cylinder 50-H.-P. engines and one 100-H.-P. engine. These engines were run for more than twelve years. By reason of the subdivision of offices, additional lamps were required, and the load was increased. Dynamos of 50% greater capacity were substituted for the old generators, and the plant has been running with complete satisfaction for three years.

When the installation becomes very large, the 50% unit can be divided into two or more machines, and the spare unit made equal to one of the divided units. To illustrate: If it is assumed that the total installation is equal to 200 E. H.-P., the writer would install three units, each of 100 E. H.-P., made up of a single-cylinder engine and generator to run for a short time at 50% over-load and for several hours at 25% over-load, one unit to be operated from 50 H.-P., or 25% of the total load, to 125 H.-P., or, say, 63% of the total load, with a possible further load up to 150 H.-P., or 75% of the total load.

The summer-day load might run from 50 to 100 H.-P., with a probable increase to 150 H.-P, under thunder-storm conditions. The unit

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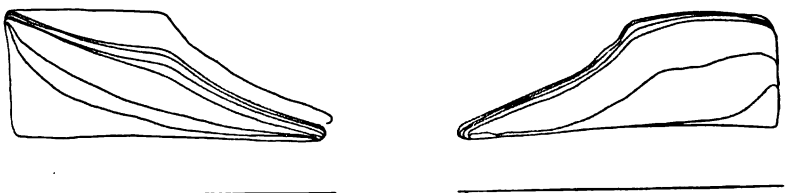
Mr. Darrach. could run at this load long enough for the engineer to put the second unit in commission.

Running under these conditions, a steam consumption of, say, 43 to 45 lbs. per hour per E. H.-P., when running at one-fourth of the entire load, or 50 H.-P.; from 34 to 36 lbs., when running at 100 E. H.-P.; and from 35 to 36 lbs., when running at 125 E. H.-P., has been obtained.

The economy is usually less than indicated by these figures, especially for smaller engines, in which the steam may run as high as 50 to 60 lbs. per E. H.-P. per hour.

Operating the machinery in this way, the writer would alternate the two units throughout the day, to prevent overheating the generators, and if the load approached 125 H.-P. he would put both units in commission.



INDICATOR DIAGRAMS.

HIGH-PRESSURE CYLINDER, LAIDLAW-DUNN-GORDON PUMP.
ONE HIGH-PRESSURE CYLINDER 17¾ INS.
TWO LOW-PRESSURE CYLINDERS 20 INS.
STROKE 24 INS.
PUMP COMES TO A FULL STOP AT INTERVALS OF LESS THAN 1 MINUTE.

FIG. 9.

If, as Mr. Bolton suggests, there is a possibility of the total maximum load being suddenly thrown on, both units (or 100% of the load) must be in commission, even if the ordinary running is at the minimum load, unless a storage battery or a connection with the outside illuminating company is provided.

If compound, non-condensing engines are installed, the writer would generally increase the size of the engine so as not to overload more than 10% when running at 75% of the total installation, although some compound engines can be run to 50% above this point of most economical operation.

The writer calculates that, if compound engines as described are used, running at 50 E. H.-P., the steam consumption would be from 38 to 39 lbs. per hour per E. H.-P.; when running 100 E. H.-P., from 30 to 31 lbs.; and when running 125 E. H.-P., from 28 to 29 lbs.

When it is considered that during at least six months of the year but little, if any, exhaust steam from the electric light plant is utilized

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for heating, and that during the remaining six months some may be wasted, the economy of high-duty engines cannot fail to be recognized, even though the first cost is greater. Mr. Darrach.

On page 41, Mr. Bolton criticises the writer's comparisons between the hydraulic and the electric elevator, and illustrates by indicator diagrams the varying load imposed by an electric elevator. The pair of indicator diagrams in Fig. 9 are taken from the high-pressure cylinders of a Laidlaw-Dun-Gordon pump, with high-pressure steam cylinders, 17½ ins. diameter, and two low-pressure cylinders of 20 ins. diameter, and 24-in. stroke. The building in which this pump is located has ten elevators, running through seventeen stories. A second pump, of the same design, with 16-in. and 18-in. cylinders, and 24-in. stroke, was running at the time the cards were taken, thus dividing and equalizing the load. The two pumps delivered into two pressure tanks, 6 ft. in diameter and 22 ft. long.

Notwithstanding this installation, and with the advantage of the storage in the pressure tanks, each pump came to a dead stop at intervals of less than a minute, so that the cards do not show the disadvantage under which the pumps were operating; but it will be noticed that the variation in load was much greater in these pumps operating the hydraulic elevator than the variation in power on the engine driving the electric elevator, even when four cars were started together, and without the intervention of a storage battery.

The writer has stated, and shows by diagram made from actual tests (Fig. 10), that the economical range of efficiency of a steam engine is from a 40 to 50% under-load to a 50% over-load. The description of the indicator diagrams, shown in Fig. 2, taken from an engine driving a dynamo direct to electric elevators, is as follows:

"Indicated horse-power, by largest card..... 154
" " " by smallest card..... 54
" " " average..... 108"

So that, if a properly rated engine of 108 H.-P. had been used in this instance, the range of economy would have been from 50% under-load to 50% over-load.

As the writer has noted, a compound, non-condensing engine, properly rated, will give an economy of from 23 to 24 lbs. of water per indicated horse-power; and the economy, within the range of operation recommended, will vary about 10% above that point. The break horse-power will run about 10% higher, or, say, 25 lbs. of steam per break horse-power-hour.

From tests made with a Laidlaw-Dun-Gordon pump, the economy per indicated horse-power runs from 24 to 26 lbs. of water, and the break horse-power will reach 30 lbs. or more; so that there is a decided advantage and economy in the steam unit operating the electric power over that operating the pumps.

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Mr. Darrach. With a Sprague electric-screw elevator, the writer has found that the starting current will vary from 50 to 100% above the regular hoisting current, and that the excessive starting current is largely governed by the car operator. This could be remedied by simply adding a starting contact button in the car.

Electric elevators recently installed in Philadelphia show a starting load not over 25% in excess of the running load, and it is simply a matter of modification of the mechanism to make the starting current much lower even than this. The writer would not advise the installation of an electric elevator plant as usually constructed without a storage battery, but the great cost of a storage battery may be discounted. In making an estimate for a plant which contemplated the use of a storage battery, it was found that, to get practically the same results, by simply changing the period over which the battery was to be used as storage (without operating the generators), the cost varied from $4 000 to $16 000.

Some three years ago, in obtaining proposals, the writer found that a hydraulic elevator plant (in which the pumps were operated by electric motors and no storage battery provided) cost as much as a Sprague screw-elevator plant, including a storage battery which cost $18 000. The latter plant was adopted. The space occupied by the storage battery was less than would have been required for the pressure and return tanks for the hydraulic elevators, with the additional advantage that all the machinery could be closed down at night and on Sundays and holidays. This Sprague plant has been in operation for two and one-half years. There has been some trouble with the traveling nut, one of the screws being of softer metal than it should have been, but it will probably last another year. The other two screws, in all probability, will last at least two years more. The electrical connections and contacts have given no trouble. Beyond this, there is but little wear and tear, and it is the writer's opinion that, on the question of repairs, this will compare favorably with any high-duty hydraulic plant.

The electric-drum elevator has been relegated to apartment houses and small office buildings. The writer sees no difficulty in such a modification of this style of machine for all kinds of service.

As far as the matter of economy in operation is concerned, the writer can give no data as to the hydraulic machines; the information obtained from tests usually made cannot be taken as a fair indication of their every-day working economy, as the elevators under test conditions are generally run continuously at their utmost speed.

With the Sprague plant noted, the writer finds that the average economy, running through a period of thirty days, varied from 3½ to 4 kilowatt-hours per car-mile. If, for the sake of argument, this is increased by 25%, to allow for the losses in the generator, in the stor-

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Mr. Darrach.

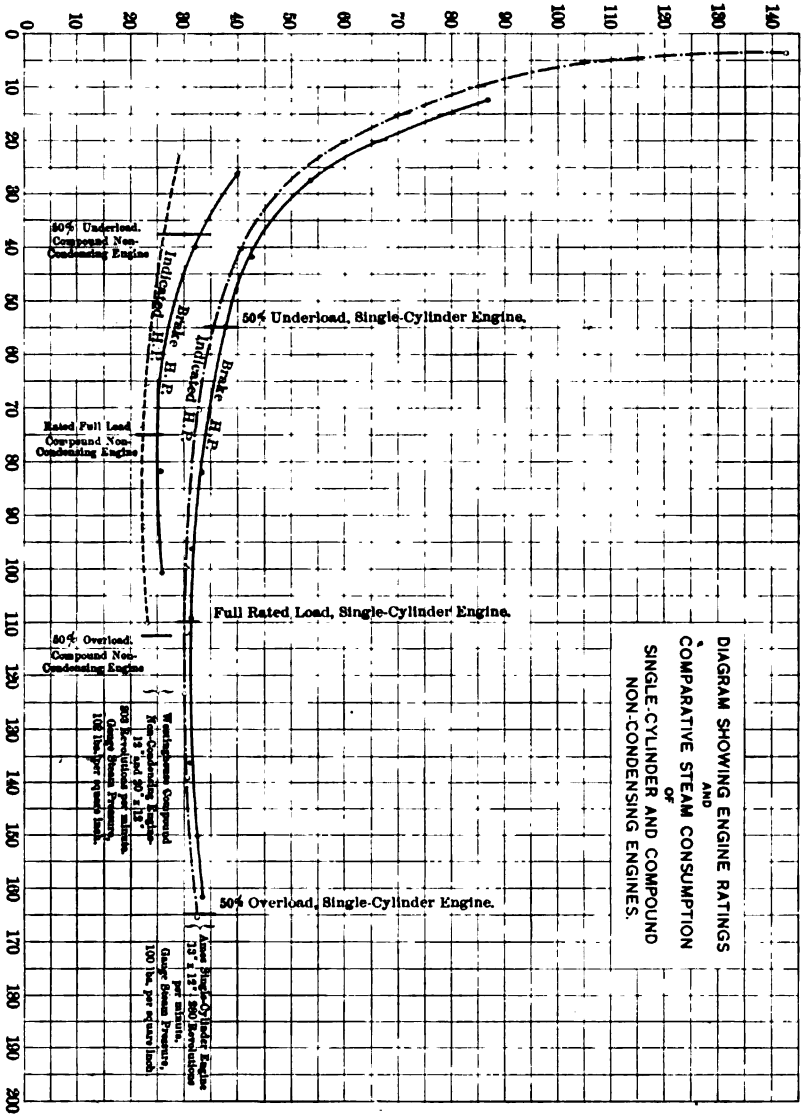

FIG. 10.

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Mr. Darrach. age battery and in transmission, the result is 5 kilowatt-hours, or 6.7 horse-power-hours, per car-mile; say, 7.4 break horse-power-hours on the engine per car-mile, or, with a compound engine, 20 lbs. of coal per car-mile.

Another advantage in using an electric elevator and battery is that by its use only one, or at most two, generator power units need be provided, and, as indicated in the paper, all machinery and lights can be electrically operated. By this method there is a decided economy, especially in the smaller apparatus; for instance, a house pump having a capacity of only 60 galls. per minute showed an economy of 75% from wire to water in the tank, which means that this small pump could be driven with from 35 to 40 lbs. of steam per horse-power-hour; but, if driven directly by steam, it would probably have used over 100 lbs. of steam per horse-power-hour.

Another advantage to be gained is in the fact that the multiplicity of steam and exhaust and drip-pipes, with their numerous joints and valves to serve the various units, is avoided, first cost and expense in maintenance is reduced, and the comfort of the engine-room is enhanced.

Referring to the hydraulic elevator, Mr. Bolton states:

"Another element, always forgotten in this connection, and interesting to the hydraulic engineer, is that, if, as stated by the author, the same quantity of water is used in a hydraulic cylinder for no load, or for a light load, as for a full load, it must not be forgotten that, with this water and the lighter loads, the car goes faster. If the water is turned into the cylinder, and the resistance of the load is very little, the car runs that much faster, and that is an improvement in the elevator service. If the load consists of only two people, and they are to be taken nearly to the top floor, the round trip can be made quickly, and in that way the car is sooner in service again. The mileage gained has to be considered as an advantage. It is not a loss, although, on the face of it, it may appear to be so."

It is evidently impossible to take advantage of this condition, for the elevators would then be run too fast, and the practice would be immediately discontinued.

Outside the question of style of pump, the great loss in the hydraulic elevator, operated with only one pressure, is that the average loads are much less than the maximum load at schedule speed for which the machine is designed, and the machine therefore runs at a loss; for, as heretofore stated, the excessive speeds which might be obtained with light loads are absolutely inadmissible. With the electric elevator, the great loss of economy is due to the starting loads, but if the average consumption of power per car-mile is considered, and the losses in the storage battery and the electric generator are added, from the writer's observation, the electric elevator compares favorably with the hydraulic machines usually installed.

If the pumping engine for the elevator service has a higher average economy than the engine driving the electric generator, the hydraulic

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machine certainly has that much to its advantage; and, as heretofore stated, the writer agrees with Mr. Bolton that more attention should be paid to the installation of high-duty engines and pumps. With regard to the questions of repairs and attention, the discussion is difficult, as much depends upon the personal equation of the operating engineer. Mr. Darrach.

The writer has called attention to the large starting currents used by electric elevators, which require the use of a storage battery, and he does not contend that a differential hydraulic elevator system with high-duty pumps may not be more economical in steam consumption than an electrical apparatus, but when there is taken into consideration the fact that two sets of hydraulic cylinders, two sets of pressure tanks and two sets of pumps, with the multiplicity of pipes, valves, etc., required, the first cost and maintenance will go far to eliminate any difference in possible steam economy.

The average speed noted in Table No. 3 includes time occupied in stops, not the maximum speed of the car. As Mr. Bolton states, a maximum speed of 700 ft. per minute is too high. In the writer's opinion 500 ft. would give more general satisfaction, and a speed of 600 ft. should not be exceeded.

Cars with less than 25 sq. ft. of floor area are not desirable, and little is gained by having an area of more than 30 sq. ft. In the Drexel and Stephen Girard Buildings (Philadelphia), there is no complaint except that the cars are crowded.

On page 14, referring to elevators, the writer calls attention to certain necessities to provide against accident. He now calls attention to certain practices that should be avoided. The counterbalance should never be run in the elevator hatchway, and the elevator should be designed so that it would be impossible for the car to be raised without the direct application of power.

Philadelphia laws require that the hatchway doors shall be closed by the operation of the starting lever. This requirement is worse than wrong; the hatchway doors should invariably be closed by a device independent of the starting mechanism.

As an illustration of the viciousness of this device, the following instance may be given: A passenger was leaving the car, the pilot-valve leaked, the car settled away from the platform, the doors suddenly closed, caught the passenger by the head, and held him prisoner. Had it not been for his presence of mind he would have been decapitated.

The premature closing of the elevator doors in this building became so notorious that the tenants made a practice of taking a running jump when leaving or entering the car.

These same laws have also required the installation of an air-cushion of a depth not less than one-sixth of the rise of the car, willy-nilly,

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Mr. Darrach. thereby imposing untold hardships upon owners of buildings in which the aforesaid device had not previously been constructed. In one instance, where the rise was about 500 ft., the air-cushion was over 80 ft. high, and cost $25 000, whereas the cost of the elevator installation itself was only $10 000. In the writer's opinion, the air-cushion is a panacea for improper design, poor construction and negligent supervision.

In the days before the height of office buildings was more than 10 stories, the construction of an air-cushion was without serious objection or expense, but when it becomes so deep that it is forced above the first floor, or into the working travel of the car, the objections become serious, involving expensive construction, loss of speed, and unnecessary expenditure of power, heavy and small hatchway doors, discomfort to passengers from air currents, and danger should the car be caught in the air-cushion by the operation of the safety brakes.

If the air-cushion is extended below the working travel of the car, the hoisting apparatus must be designed to operate to the full depth of the air-cushion; and if this air-cushion is 30 to 40 ft. below the first floor, it is not only expensive in construction but becomes a receptacle for all sorts of filth and trash; in fact, it may be the cause of greater loss of life than it could possibly save. As the writer has stated, an air-cushion of sufficient depth to stop the car in case of a runaway is all that may be considered as a necessity.

The rupture of the suspending cables (the only legitimate reason for the need of an air-cushion) need never happen, and is a most remote contingency.*

To determine the advisability of installing an electric storage battery: In comparison with the first cost, losses in operation, and expense of maintenance, and the value of the space the battery would occupy, it must be considered that one generating unit may be omitted, that the units may be reduced to the average load, and that they can be operated at their highest economy; also, the convenience in having a storage to draw from on all conditions of the variable demands; and that, if the battery is large enough, engines need not run at night or on holidays. The result of such comparison will vary with each building.

The writer is not prepared to admit that direct pumpage of the water supply is better under all conditions than the tank system, any more than he would be prepared to abandon the use of reservoirs for a city supply. If the building is so high that a differential system

* Recently there has been placed on the market what is known as the "Cruikshank Safety Device," and the writer understands that it has been accepted by the Philadelphia authorities in lieu of an air-cushion. This device consists of a number of metallic wires, running on either side of the elevator shaft from top to bottom. Between each floor "retarders" are strung in these vertical wires. The "retarders" are made of a pair of metallic plates, spaced the fraction of an inch apart, with staggered rivets around which the wires pass. The car is furnished with dogs which engage the "retarders" at a prearranged overspeed, and the car is slowed down by the friction of the retarder against the wires. If the car has not stopped by reason of the friction of the first retarder, it brings the second into action, and so on until the car is finally brought to a standstill.

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would be economical, it would be advisable to provide storage tanks for the upper service, with a connection to the lower service controlled by a pressure-regulating valve. With such a plant, continuous pumping at night and on holidays would be unnecessary, and, as the pumps delivering to the tanks could be comparatively large, and could be operated with a steady load, their economy would compare favorably with the low direct-pressure system. Mr. Darrach.

If it were not for the great cost of installation, and for maintenance and operation, every plant would be provided with an electric storage battery. These objections do not obtain in providing storage for water, and yet the same purpose is served.

The remarks on saving water are very important. As stated, the self-flushing urinal is the source of enormous waste, and there is no reason why the pull-chain device, or a small flushometer, should not be installed. The reason the municipal authorities object to it is no doubt the fear that it will not be used. The same fear might force the adoption of an automatic flushing device for each water-closet bowl.

The condensing water from the refrigerating apparatus should certainly be used again, and can be used for the warm-water supply to the offices, feed-water for the boilers, and for flushing water-closets and urinals. In fact, all known precautions should be taken to prevent waste.

The idea of utilizing the waste heat in the drip-tank has presented itself to the writer, but generally for nine or ten months of the year there is enough exhaust steam to heat all the water and the building; so that it becomes a question whether or not the cost of installation and the expense of maintenance in utilizing the drip-tank heat is always advisable.*

Referring to the items of necessities, "Policing and Fire Protection," the writer did not have in mind the thought advanced by Mr. Bolton:

"What the author describes as 'policing,' consists in having an ex-policeman acting as hall-man in the main hall, a matter that is not a mechanical subject, and, in New York City, is usually accomplished very easily."

The general question of watchman's service during the night and on holidays is very important, especially when there is a restaurant in the building, and its employees are not under the control of the office building management. To ensure fidelity, a watchman's clock, connected with stations located at convenient points throughout the building, should be installed, as well as a system of fire-alarm signals connecting each floor with annunciators and call-bells in both the chief engineer's

* Illustrating one use of the drip-tank, not originally contemplated, and at the same time the ingenuity of the operating engineer: Some years ago, during a blizzard, the 20 000 sq. ft. of pavement around the Drexel Building was covered with some 3 ft. of snow. To meet the emergency, Mr. John Frigar, the Chief Engineer and Superintendent, shoveled the snow into the drip-tank, and pumped the water into the sewer, which was the quickest and most economical method to dispose of it. It might be well in the future to take advantage of such an idea, and so locate the drip-well that a portable chute would provide an easy means for conducting the snow to it.

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Mr. Darrach. office and the engine-room. There is no doubt that such a system would have prevented the fire in the German-American Building mentioned by Mr. Bolton.

Tenants frequently leave their lamps burning, their windows open, and their doors unlocked, and although it may be said that the members of the cleaning force would naturally attend to these matters, they may be derelict.

A serious fire starting in the building during office hours is hardly probable. The main care must be at night and on holidays. In this connection, it would be advisable for the superintendent or chief engineer to institute a fire-drill with his employees, so that, if there should be an alarm of fire, confusion would be avoided and prompt service rendered.

In Philadelphia, the Board of Fire Underwriters requires what is called the "Fire Underwriters' pump," and a system of pipes, nozzles and hose throughout the building. To make this effective at night, an engineer should always be on duty, and the fire-pump ready for instant service, which, unfortunately, is not always the case. An electrically-driven fire-pump, with connection to the service of the illuminating company, or to the storage battery (if such is installed), would be better than a pump driven by steam, as such a pump could be more quickly put into service.

In some cities a separate system of water service under high pressure has been installed for extinguishing fires, and this practice seems to be extending. The City of Philadelphia is now constructing a very extensive system of this kind, which will cover practically all of the business district. The water-supply pipes for fire-service through the building should be directly connected with this municipal fire protection.

In the writer's opinion, the number as well as the size of the fire-service pipes throughout the building, and the connections to the municipal supply, should be proportioned, not only in reference to the height of the building, but also to its floor area; and the fire-nozzles should be located on each floor, so that hose of over 100 ft. in length would be unnecessary.

The writer has called attention to the necessity of providing chemical fire-extinguishers. Such a provision will be generally sufficient to extinguish incipient fires starting in the building, but this is not the only danger from fire to which the building is subject. Fires in adjacent buildings may really be more cause for danger than a fire starting in the building itself, and an ample water service under high pressure in the building could be utilized to protect it from such exterior fires.

Mr. Bolton says:

"Office buildings might be provided with a chemical extinguishing system, with a nozzle and valve in every room, or on every floor; then,

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if a fire starts in any room, close the room, or the floor, and turn on the chemical extinguisher, or even fill the room with steam, as is now done on shipboard." Mr. Darrach.

This seems to be somewhat of a gigantic scheme, and difficult to accomplish. By reason of the elevator shafts running through each story, it would often be impossible to isolate that story from the rest of the building, and it hardly seems necessary to provide a nozzle and valve in every room. Again, to prove effective, the entire system of pipes must always be charged with the extinguishing gas, be it carbonic acid or steam.

The writer's idea is that a number of portable fire-extinguishers should be distributed in each story, or possibly an apparatus on wheels for each floor. If deemed advisable to go so far, each office, or such rooms as might be considered extra hazardous, could be furnished with a portable chemical fire-extinguisher; or there might be provided, in a convenient rack for instant use, several packages or vessels containing a fire-extinguishing preparation.

Instead of providing outlets from a chemical or steam fire-extinguishing system in each room, a system of thermostats might be installed, with one thermostat in each room, connected with annunciators in the chief engineer's office and the engine-room.

In this connection, it might be said that one principal cause of fire is dirt and rubbish, which by proper police service could be prevented.

For protection from fires outside of the building, the Philadelphia Board of Fire Underwriters requires a water supply under pressure, with fire-hose and nozzles on the roof.

During a recent fire at Twelfth and Market Streets, Philadelphia, the "Policing and Fire Protection" system of the Stephen Girard Building, not only saved the building from great loss, but also proved such a help to the City Fire Department in extinguishing the outside fire as to call for unqualified praise from the public press.

But after all has been done, the importance of the position of chief engineer of the modern office building cannot be over-estimated. In connection with his mechanical duties, he is often the general superintendent, and he becomes an executive of no mean order. This position opens a field for technical graduates, who, after an experience in the practical operation and management of an office building, could not only improve the service, but enjoy a salary which the crowded state of the profession would otherwise prevent.

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* Professional ethics, as well as parliamentary law, is intended to prevent altercation and unnecessary waste of time, so that whatever is said or done in the discussion of a subject may be of benefit. All calm argument based upon facts tends to our enlightenment, but there

* Discussion continued by letter, after reading that part of Mr. Bolton's discussion beginning on page 43.

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Mr. Darrach. is a wide difference between discussion and contention. The first is scientific and deals with facts, or at least honest opinion, while the latter is personal and may be marred by *ex parte* feeling, misconception or misstatement. Widely divergent opinions may be stated so as to be interesting without being offensive.

If these thoughts are correct, and if they had been followed, the last paragraph, *in re* "general illumination," on page 44, would never have been written.

By reference to the remarks upon the subject under contention (page 56), it will be found that the words "either" and "or" were used, and not the word "and."

By reference to Fig. 8 and the description of Curve 3 (page 54), it will be noted that in the contention (page 44) the statement: "On the downward-supply system, with a vacuum return, the risers need be of no larger size than those the author appropriates to his own method." is but a truism, copied directly from the statement which is sought to be criticised.

This criticism is incomplete, as it does not say whether or not the vacuum system is supplied from both above and below. In the double-supply system the risers are reduced in size to one-half the area, as shown in Curve 4. No doubt this fact was lost sight of in the heat of contention.

It is not necessary to discuss further the advantages of the double-supply system, or to compare it with one which would require a system of horizontal supply mains of maximum dimensions erected in the middle story of the building.

Artificial ventilation of the office building seems to meet with approval either directly or indirectly in all the discussions, consequently the factor, N, in the formula, is necessary to provide against "stagnant" air, and proves that there is a relation between the cubic contents of the building and the boiler power required—although this relation has a somewhat wide range—as stated in the paper on page 2.

The relative economic consumption of power in electric and hydraulic elevators, the comparative cost of construction and operation and application for special conditions and specific purposes, has been fully discussed elsewhere, and, for the present, it seems useless to pursue the subject further; but, in the writer's opinion, enough has been said to show that, with the hydraulic elevator, complicated machinery, large storage of power, or both, are required to obtain economical results that will compare with the electric elevator, even of a type probably most deserving of criticism.

In closing this discussion, the writer acknowledges the courtesy extended to him by the following gentlemen, and the opportunity given to obtain many of the data in the original paper and in the discussion:

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Mr. John Frigar, Chief Engineer, Drexel Building, Philadelphia; Mr. Samuel Dinsmore, Chief Engineer and Superintendent, Stephen Girard Building, Philadelphia; Mr. Alonzo Dalton, Chief Engineer and Superintendent, Land Title and Trust Company's Building, Philadelphia; Mr. George Atkins, Chief Engineer and Superintendent, and Mr. W. G. Rice, Assistant Engineer, Real Estate Trust Company's Building, Philadelphia; Mr. William A. McEwen, Superintendent, and Mr. Wm. H. Hewitson, Chief Engineer, United Gas Improvement Company's Building, Philadelphia. Mr. Darrach.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

No. 917.

A GRAPHICAL METHOD FOR THE SOLUTION OF STRESSES IN THE CONTINUOUS GIRDER, AS APPLIED TO DRAW-BRIDGES.*

By George F. Barton, Esq.

With Discussion by Messrs. C. H. Lindenberger, H. A. La Chicotte and George F. Barton.

A few years ago a graphical method for the solution of the continuous girder was brought to the writer's notice by C. R. Grimm, M. Am. Soc. C. E. The method is so simple and comprehensive, and yet seems to be almost entirely unknown to American engineers—although used by German designers—that the writer has felt it almost a duty to bring it to the attention of his countrymen.

Of course, the only use for the solution of the continuous girder in this country is in designing draw-bridges, and, while this method may be made to give moments and shears for any kind of loading, and with a constant or variable moment of inertia, or for a truss span, where the moment of inertia would not be used, but each individual member be treated by itself, all this would complicate the explanation to such a degree that it would be entirely out of place anywhere except in a textbook, and the writer believes that the whole discussion is given by Professor Heinrich F. B. Müller-Breslau, of the Technical High School of Berlin.

* Presented at the meeting of November 20th, 1901.

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In this country, in draw-bridge work, it is generally assumed that the bridge has a constant moment of inertia throughout.

The formulas, also, are based on the assumption that the bridge, whether it be a truss or girder, is a beam, and will deflect like a beam.

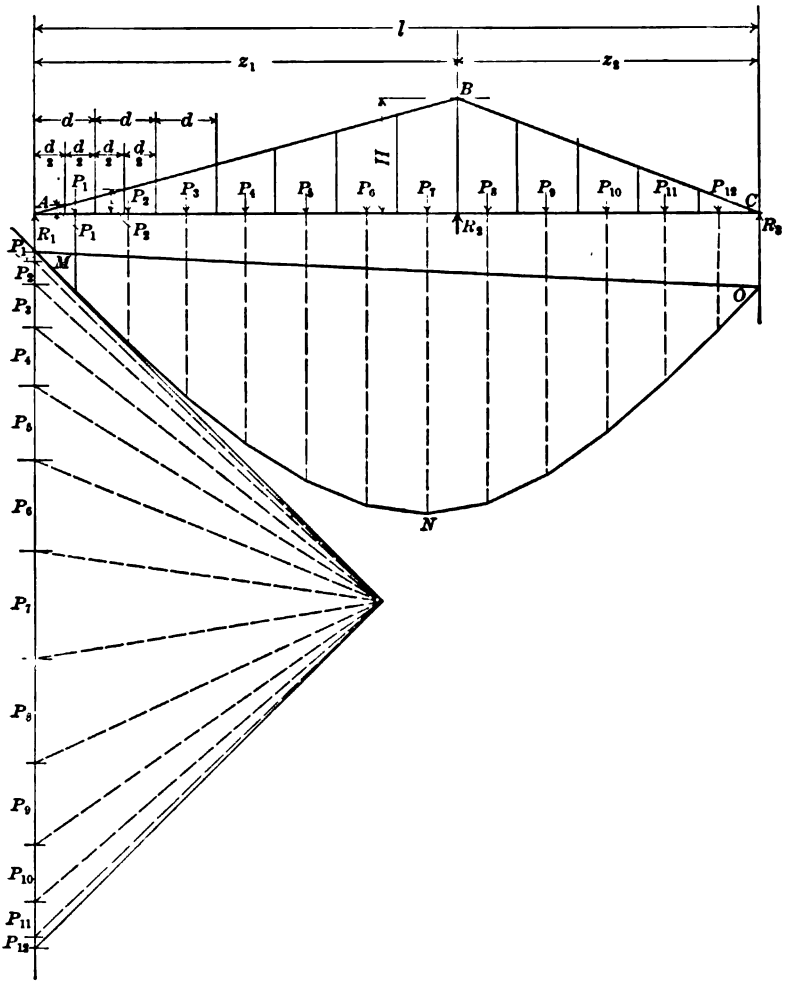

FIG. 1.

As before mentioned, the following method might be made to give the moments and shears directly from the diagram; but if the reactions are once found for any given loading it is a very simple matter to find the resulting moments and shears analytically, and in this case it is

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easier and quicker. For this reason the following graphical method will simply be a means to find graphically the reactions which may result from any system of loading that may be applied to the structure.

This method is so simple that a bridge may easily be designed under a system of moving wheel loads, and it makes no difference whether or not the arms of the structure are of the same length.

The writer thinks the foregoing conditions of loading and of unequal arms, to be designed by the ordinary analytical methods used in this country, would give most designers a very tiresome task, and they would never feel certain that they had not made some mistake in their figures.

Some of the advantages to be gained by using the following method may be briefly stated :

One diagram, without any change, will be sufficient for all bridges with equal arms, no matter what the system of loading or what length the arms may be.

All the results being scaled from the diagram, the designer has a picture before him, and his results may be checked in a few moments with absolute certainty that he is correct.

The method will be given first and the explanation afterward.

Assume the case of a bridge with unequal arms :

In Fig. 1, let z_1 and z_2 be the lengths of the two arms. Draw a triangle, *A B C*, the altitude of which is any height, *H*, and the vertex of which, *B*, is above the intermediate support, R_2. Now assume that this triangle constitutes a loading on a beam the length of which is $l = z_1 + z_2$. Then draw an equilibrium polygon, *M N O*, using the system of loading represented by the triangle *A B C*. This triangle may be divided into any convenient number of equal or unequal parts as *d*, *d*. Let the areas of these strips be called P_1, P_2, P_3, etc., and the lines of action of the loads be vertically through their center of gravity.

The load line is laid off by making P_1, P_2, P_3, etc., equal to these areas. The pole distance may be any length, and the equilibrium polygon, *M N O*, be thus constructed.

Of course, the narrower the strips, the more accurate will be the work; but care should be taken to construct a load line which is not too long; also, for the same reason, the altitude *H* should not be assumed too great.

If all the strips are made of the same width, their areas will be

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proportional to the length of their middle ordinates, and these can be used to represent the loads as well as the actual areas of the strips.

Having drawn the polygon, $M\ N\ O$, draw $O\ N\ Q$ and $M\ N\ S$ (see Fig. 2). Now, if there are loads P_1 and P_2 on the bridge

R_1 would equal $\dfrac{P_1\,b - P^2\,f}{C}$

R_2 " " $\dfrac{P_1\,a + P_2\,m}{d}$

and R_3 " " $\dfrac{-P_1\,g + P_2\,h}{k}$

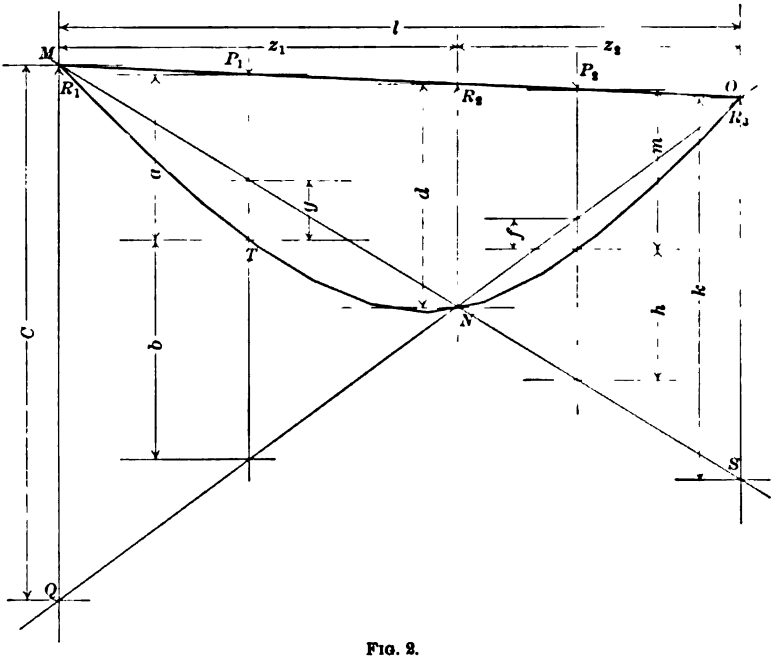

FIG. 2.

It is evident that all loads in spans z_1 and z_2 tend to increase R_2; but those in z_1 tend to increase R_1 and decrease R_3, as shown by the negative ordinate g, that is, the ordinates between the curve $M\ T\ N$ and the line $M\ N$ show the uplifting effect on R_3, due to the loads in span z_1.

The application of the diagram consists in placing the loads on the bridge so as to give the maximum moments, or shears, as the case may be, and scaling the ordinates under the loads. Suppose R_1 was wanted, for any system of loads in both spans.

Measure, with any scale, each ordinate b, for loads in span z_1, under

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each particular load, and multiply the ordinate by the load, add together all these products for span z_1; then scale each ordinate f, for the loads in span z_2, under each particular load, and multiply the ordinate by the load; add together all these products for span z_2 and subtract them from the sum of the products for span z_1; divide the remainder by C and the result will be R_1.

R_2 and R_3 may be found in a similar manner, as explained in Fig. 2.

After the reactions have been found it is a very easy matter to get the resulting moments and shears.

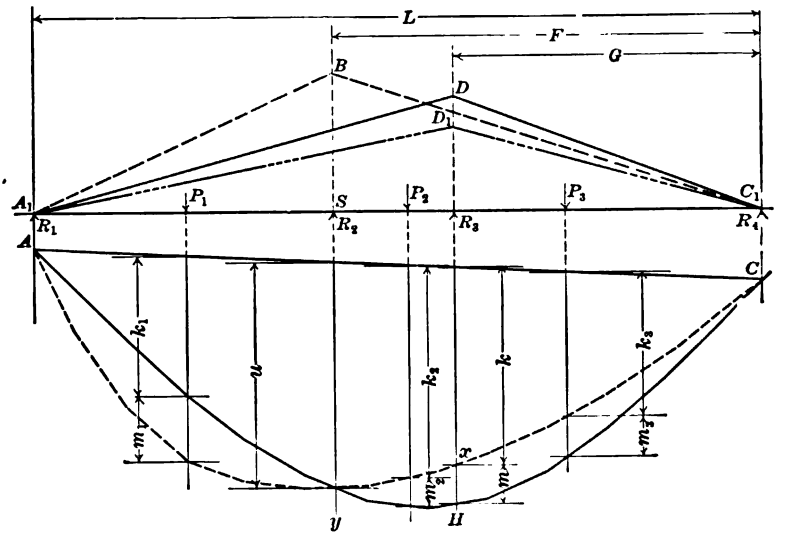

FIG. 3.

For a Span over Four Supports.—In Fig. 3, if R_2 and R_3 be known, it is a comparatively simple matter to find R_1 and R_4 analytically. Although R_1 and R_4 might be found graphically, the method is rather complicated and will not be given.

In order to find R_3 construct an equilibrium polygon from the loading A_1 B C_1, the altitude B S being any convenient height, then construct another equilibrium polygon from the loading A_1 D C_1, making it pass through A, y and C.

Suppose it be required to pass an equilibrium polygon through A, B and C (Fig. 4). Lay off the load line, making

$$F\,G = P_1;\ G\,J = P_2;\ J\,L = P_3, \text{ and } L\,M = P_4.$$

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Take any point, as O, for a pole, and construct the equilibrium polygon $A\ E\ D$, starting at A, any one of the given points. Now, as the required polygon must pass through C, if $C\ D$ is drawn parallel to the resultant of the forces acting on the beam, the same reactions for the beam will result, whether it be supported at A and D, or A C. These reactions, therefore, can be found by drawing the closing line $A\ D$, and, from O, drawing $O\ K$ parallel to $A\ D$. Then the re-

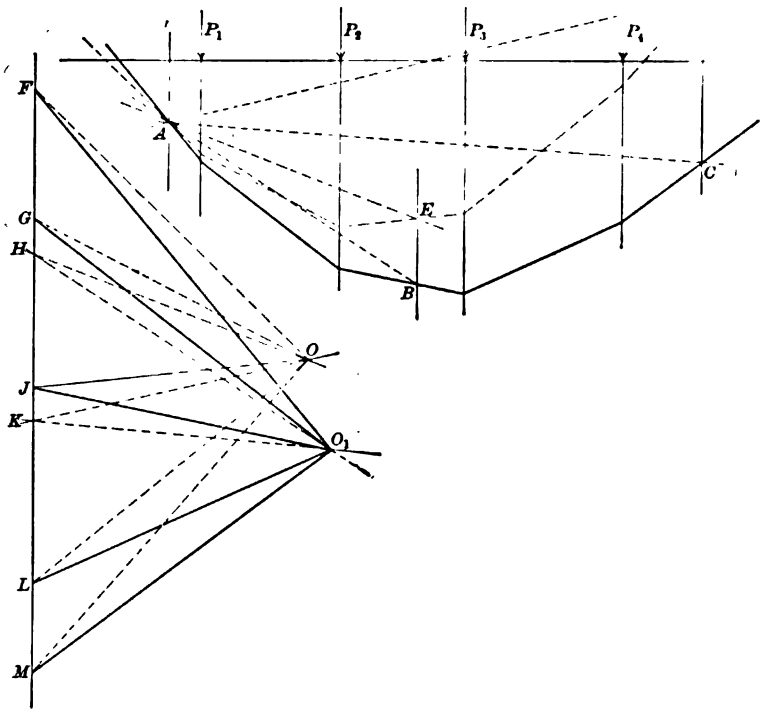

Fig. 4.

actions at A and D will be $F\ K$ and $K\ M$, but these are also the reactions at A and C; it is evident, therefore, that the pole of the force polygon must lie somewhere on a line $K\ O_1$ through K drawn parallel to $A\ C$. Now, through B draw $B\ E$ parallel to the resultant of P_1 and P_2; then the reactions at A and E are the same as those at A and B, for a beam carrying P_1 and P_2 and supported at A and E, or A and B. The reac-

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tions at A and E are obtained by drawing $A\ E$, and making $O\ H$ parallel to it through O; then $F\ H$ equals the reaction at A, and $H\ J$ the reaction at E; but these are also the reactions at A and B, and it is evident, therefore, that the pole of the force polygon must lie somewhere on a line $H\ O_1$, drawn through H parallel to $A\ B$; but it has already been shown that O_1 must be somewhere on $O_1\ K$, and, therefore, it must lie at the intersection of $O_1\ K$ and $O_1\ H$. This now gives the location of the pole of a force polygon which will produce an equilibrium polygon passing through A, B and C.

Suppose the loads P_1, P_2 and P_3 are in the span, then R_3, due to these loads, would be (see Fig. 3)

$$R_3 = \frac{-P_1 m_1 + P_2 m_2 + P_3 m_3}{m},$$

that is, all loads on the left of R_2 tend to decrease R_3, as shown by the negative ordinates m_1, while the loads to the right of R_2 tend to increase R_3, as shown by the positive ordinates m_2 and m_3.

To find R_2, draw another equilibrium polygon passing through A, x and C, the loading being the triangle $A_1\ D_1\ C_1$, and proceed as for R_3.

Having found R_2 and R_3, R_1 may be obtained by taking the moments about R_4.

$$R_1 = \frac{\Sigma \text{ Moments of } P_1,\ P_2,\ P_3, \text{ etc., about } R_4 - R_2\ F - R_3\ G}{L}.$$

The explanation of the foregoing method is due primarily to two facts.

The first is, that if the deflection of the beam $A\ B$ at u is y, due to the load P at m, and the load P be moved to u, then the deflection at m will be y (see Fig. 5). This may easily be tested by means of the equation of the elastic line, or, as the equation of the elastic line is derived primarily from the equation of the bending moment, it might be stated that if the moment at u with P at m equals the moment at m with P at u, then the deflections at these points will also be alike.

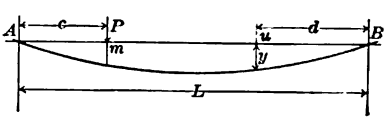

FIG. 5.

With P at m, the moment at u equals $\frac{P\ c\ d}{L}$, and with P at u, the moment at m equals $\frac{P\ d\ c}{L}$, which is identical with the first moment. Therefore the deflections are equal.

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The second fact is that, if the deflection curve has been drawn, it will then be possible to find the intermediate reaction for any system of loading.

In Fig. 6 suppose $A\ F\ D$ to be the deflection curve for a beam $A\ D$ resting on R_1 and R_3, with a load of unity at E.

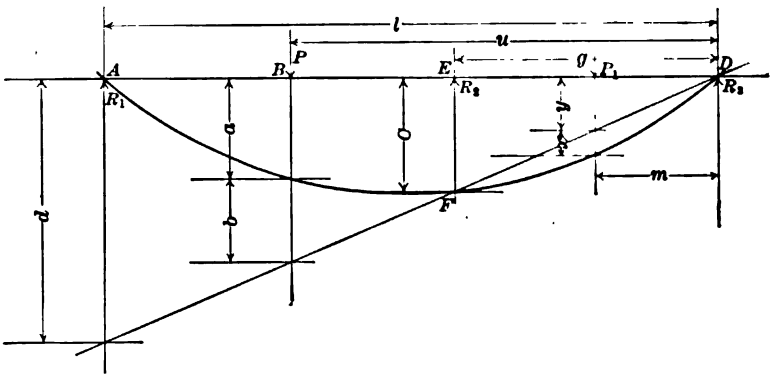

FIG. 6.

If a load of unity be placed at E the deflection is C at E; for a load of R_2 at E the deflection would be $R_2\ C$.

If a load of P be placed at E the deflection at B would be Pa.

Now, suppose a force of R_2 to press up at E just hard enough to cause $A\ F\ D$ to be a straight line; in other words, to take the deflection out of the beam at E, then $R_2\ C$ would have to equal Pa, but as P, a and C are known, $R_2 = \frac{Pa}{C}$; and, with a load P and P_1 on the beam, R_2 would equal $\frac{Pa + P_1\ (x + y).}{C}$

To make this clearer: P at E causes a deflection at B equal to Pa; P at B causes a deflection at E equal to Pa, but R_2 at E causes a deflection at E equal to $R_2\ C$; therefore, $\frac{R_2}{P} = \frac{a}{C}$, or $R_2 = \frac{P\,a}{C}$.

In order to find R_1, take the moment about D and divide by l.

$$R_1 = \frac{P_1\,m + P\,u - R_2\,g}{l},$$

but

$$R_2 = \frac{P\,a + P_1\,(x + y)}{C}.$$

Substitute this for R_2, above, and

$$R_1 = \frac{P_1\,m\,C + P\,u\,C - P_1\,x\,g - P_1\,y\,g - P\,a\,g}{C\,l};$$

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but $\frac{u}{l} = \frac{a+b}{d}$, and $\frac{m}{l} = \frac{y}{d}$, also, $\frac{g}{l} = \frac{C}{d}$;

substituting these in the foregoing,

$$R_1 = \frac{P_1 y}{d} + \frac{P a}{d} + \frac{P b}{d} - \frac{P_1 x}{d} - \frac{P_1 y}{d} - \frac{P a}{d},$$

then canceling, $$R_1 = \frac{P b}{d} - \frac{P_1 x}{d}.$$

R_3 may be found in a similar manner.

In the span over four supports (see Fig. 3), the polygon $A\,y\,x\,C$ is the deflection polygon for a load of unity at S, and u represents this deflection; now suppose a load R_3 be placed at E just large enough to cause the beam to deflect up, until the upward deflection of the beam at S is equal to u, then the deflection of the beam would be down, as shown by m_1, between A_1, and S, and up as shown by m_2, m and m_3, between S and C_1.

The value of R_3 is, therefore, $\frac{-P_1 m_1 + P_2 m_2 + P_3 m_3}{m}$.

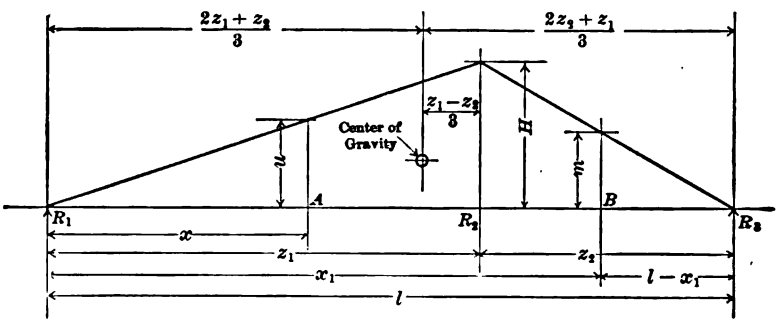

FIG. 7.

This might be explained in a different manner:

With the loads P_1, P_2, R_3 and P_3 on the beam, we would have

$$R_2\,u = P_1\,(k_1 + m_1) + P_2\,k_2 - R_3\,k + P_3\,k_3,$$

or, $R_3\,k = P_1\,k_1 + P_1\,m_1 + P_2\,k_2 + P_3\,k_3 - R_2\,u$.....(A)

also, $R_3\,k + R_3\,m = P_1\,k_1 - R_2\,u + P_2\,k_2 + P_2\,m_2 + P_3\,k_3 + P_3\,m_3$....(B)

Subtracting Equation (A) from Equation (B)

$$R_3\,m = -P_1\,m_1 + P_2\,m_2 + P_3\,m_3,$$

or, $$R_3 = \frac{-P_1\,m_1 + P_2\,m_2 + P_3\,m_3}{m}$$

Q. E. D.

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Thus it is seen that if the deflection curve could be drawn to an exaggerated scale, so that the ordinates could be easily scaled, it would be a simple matter to get the reactions for a continuous girder.

The problem now becomes: To construct, graphically, a curve which will be a true deflection curve. It has been found that this may be drawn as an equilibrium polygon, with a loading as shown in Fig. 7, in which the vertex of the triangle is over the point of application of the force.

The moment at any point, as at A, a distance of x from R_1, for this loading, will be

$$M_x = R_1 x - \frac{u x^2}{6}; \text{ but } R_1 = \frac{H l}{2 l}\left(\frac{2 z_2 + z_1}{3}\right),$$

and since $\frac{u}{x} = \frac{H}{z_1}$, and $u = \frac{H x}{z_1}$,

$$M_x = \frac{H x}{6 z_1}[2 z_1 l - z_1^2 - x^2] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Also, the moment at a point B, distant x_1 from R_1, will be

$$M_{x_1} = - R_3 (l - x_1) + \frac{m (l - x_1)^2}{6}; \text{ but } R_3 = \frac{H l}{2 l}\left(\frac{2 z_1 + z_2}{3}\right)$$

and since $\frac{m}{l - x_1} = \frac{H}{z_2}$, and $m = \frac{H (l - x_1)}{z_2}$,

we have $M_{x_1} = -\frac{H}{6}(l - x_1)(2 z_1 + z_2) + \frac{H}{6 z_2}(l - x_1)^3$

$$M_{x_1} = \frac{H}{6 z_2}(l - x_1)\left[(l - x_1)^2 - 2 z_1 z_2 - z_2^2\right],$$

$$M_{x_1} = \frac{H}{6 z_2}(l - x_1)[l^2 - 2 l x_1 + x_1^2 - 2 z_1 z_2 - z_2^2]$$

but $z_1 = l - z_2$, then $2 z_1 z_2 = 2 z_2 l - 2 z_2^2$.

Substitute this in the above equation,

then $M_{x_1} = \frac{H}{6 z_2}(l - x_1)[l^2 - 2 l z_2 + 2 z_2^2 - z_2^2 - 2 l x_1 + x_1^2]$,

or, $M_{x_1} = \frac{H}{6 z_2}(l - x_1)[l^2 - 2 l z_2 + z_2^2 - 2 l x_1 + x_1^2]$

or, $M_{x_1} = \frac{H}{6 z_2}(l - x_1)\left[(l - z_2)^2 - 2 l x_1 + x_1^2\right]$,

but $l - z_2 = z_1$,

therefore

$$M_{x_1} = \frac{H}{6 z_2}(l - x_1)[z_1^2 - 2 l x_1 + x_1^2] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If H be made equal to $\frac{P z_1 z_2}{E I l}$ (a constant quantity), it will be seen that Equations (1) and (2) become identical with the equations of the

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elastic line for a beam $z_1 + z_2$ long, loaded with a load P, at a distance of z_1 from the left end.

As ratios, only, are used in the graphical methods previously described, it will be seen that H may be made of any length; but, when it is made equal to $\frac{P z_1 z_2}{E I l}$, the equilibrium polygon becomes the true deflection curve.

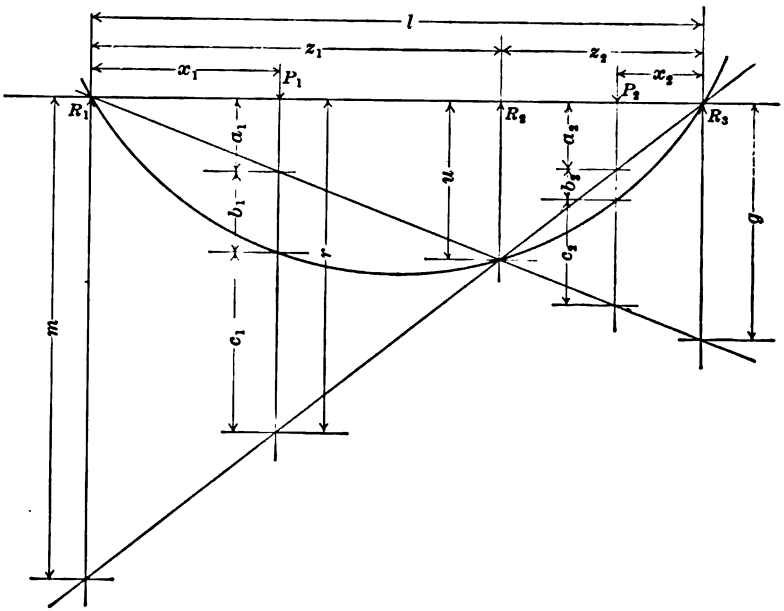

FIG. 8.

The writer offers the following set of formulas for calculating the reactions on continuous girders. They are entirely different from those in common use, but give the same results. They are simply an analytical expression for the preceding graphical method, and the writer thinks they are more easily applied. An analytical discussion is out of place in this paper, but the formulas are given because they are new and seem to possess some advantage over the old ones in use.

Starting with Equation (1), which is the value of $(a_1 + b_1) = \frac{H x_1}{6 z_1}(2 z_1 l - z_1^2 - x_1^2)$; as H may have any value, let it be called unity.

Then $(a_1 + b_1) = \frac{x_1}{6 z_1}(2 z_1 l - z_1^2 - x_1^2)$ ..... (3)

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Let $x_1 = z_1$, then $(a_1 + b_1) = u = \frac{1}{6}(2 z_1 l - 2 z_1^2) = \frac{z_1}{3}(l - z_1)$.

or, $$u = \frac{z_1 z_2}{3} \quad \ldots\ldots (4)$$

$$m = \frac{u l}{z_2}, \text{ or } m = \frac{l z_1}{3} \quad \ldots\ldots (5)$$

$$g = \frac{u l}{z_1}, \text{ or } g = \frac{l z_2}{3} \quad \ldots\ldots (6)$$

$$a_1 = \frac{u x_1}{z_1}, \text{ or } a_1 = \frac{z_2 x_1}{3} \quad \ldots\ldots (7)$$

$$b_1 = \frac{x_1}{6 z_1}(2 z_1 l - z_1^2 - x_1^2) - \frac{z_2 x_1}{3} = \frac{x_1}{6 z_1}(2 z_1 l - z_1^2 - x_1^2 - 2 z_1 z_2)$$

$$b_1 = \frac{x_1}{6 z_1}(z_1^2 - x_1^2) \quad \ldots\ldots (8)$$

$$\frac{r}{m} = \frac{l - x_1}{l}, \text{ or } r = \frac{(l - x_1) m}{l}, \text{ but } r - (a_1 + b_1) = c_1,$$

or, $$c_1 = \frac{(l - x_1) m}{l} - (a_1 + b_1) = \frac{(l - x_1) z_1}{3} - \frac{x_1}{6 z_1}(2 l z_1 - z_1^2 - x_1^2),$$

or, $$c_1 = \frac{1}{6 z_1}(2 l z_1^2 - 2 x_1 z_1^2 - 2 l z_1 x_1 + z_1^2 x_1 + x_1^3)$$

$$= \frac{1}{6 z_1}(2 l z_1^2 - z_1^2 x_1 - 2 l z_1 x_1 + x_1^3),$$

or, $$c_1 = \frac{l}{3}(z_1 - x_1) - \frac{x_1}{6 z_1}(z_1^2 - x_1^2) \quad \ldots\ldots (9)$$

Similarly,

$$a_2 = \frac{z_1 x_2}{3} \quad \ldots\ldots (10)$$

$$b_2 = \frac{x_2}{6 z_2}(z_2^2 - x_2^2) \quad \ldots\ldots (11)$$

$$c_2 = \frac{l}{3}(z_2 - x_2) - \frac{x_2}{6 z_2}(z_2^2 - x_2^2) \quad \ldots\ldots (12)$$

$$R_1 = \frac{P_1 c_1 - P_2 b_2}{m} \quad \ldots\ldots (13)$$

$$R_2 = \frac{P_1 (a_1 + b_1) + P_2 (a_2 + b_2)}{u} \quad \ldots\ldots (14)$$

$$R_3 = \frac{- P_1 b_1 + P_2 c_2}{g} \quad \ldots\ldots (15)$$

$$(a_2 + b_2) = \frac{x_2}{6 z_2}(2 z_2 l - z_2^2 - x_2^2) \quad \ldots\ldots (16)$$

In the case of uniform loads covering all or part of the span, or with a different load on each arm, it is evident that the areas will be

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required between the elastic curve and the closing line (see Fig. 9). Starting with Equation (1), and letting H = unity,

$$y = M_x = \frac{x}{6\,z_1}\,(2\,z_1\,l - z_1^2 - x^2),$$

or,

$$y = \frac{1}{6\,z_1}\,(2z_1\,l\,x - z_1^2\,x - x^3).$$

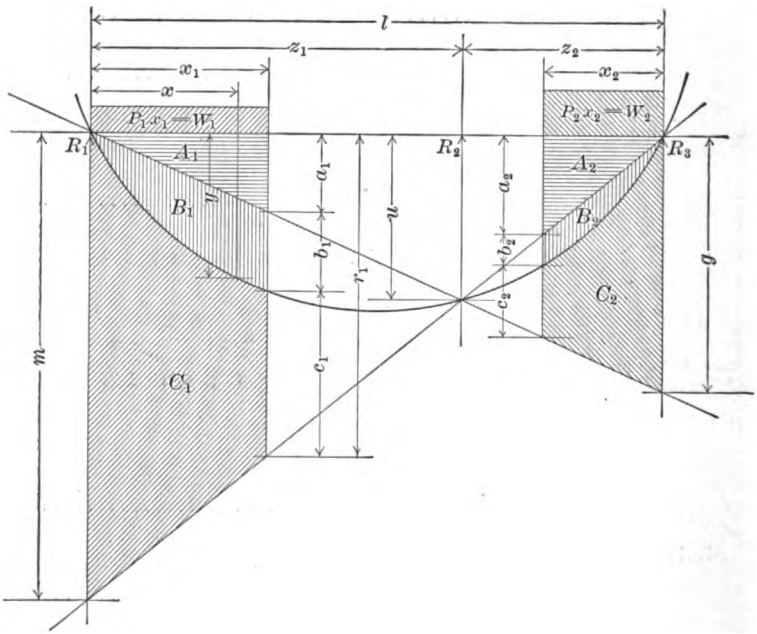

FIG. 9.

Multiply both sides of the equation by $d\,x$, then the $\int_0^{z_1} y\,d\,x =$ area bounded by the curve and the closing line $= A_1 + B_1$; therefore,

$$A_1 + B_1 = \frac{1}{6\,z_1}\int_0^{x_1} (2\,z_1\,l\,x\,d\,x - z_1^2\,x\,d\,x - x^3\,d\,x),$$

or,

$$A_1 + B_1 = \frac{1}{6\,z_1}\left[z_1\,l\,x_1^2 - \frac{z_1^2\,x_1^2}{2} - \frac{x_1^4}{4}\right],$$

or,

$$A_1 + B_1 = \frac{x_1^2}{24\,z_1}\,(4\,z_1\,l - 2\,z_1^2 - x_1^2) \quad \ldots\ldots\ldots\ldots\ldots (17)$$

But as these ordinates, y_1, must be multiplied by P_1, Equation (17) would become $\frac{P_1\,x_1^2}{24\,z_1}\,[4\,z_1\,l - 2\,z_1^2 - x_1^2]$ ..... (18)

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But $P_1 x_1 = W_1$, therefore, Equation (18) becomes

$$A_1 + B_1 = \frac{W_1 x_1}{24 z_1} (4 z_1 l - 2 z_1^2 - x_1^2) \quad \ldots\ldots (19)$$

But $A_1 = \frac{a_1 x_1}{2}$, and $a_1 = \frac{u x_1}{z_1}$, or $A_1 = \frac{u x_1^2}{2 z_1}$; but, from Equation (4),

$$u = \frac{z_1 z_2}{3}, \text{ therefore } A_1 = \frac{z_2 x_1^2}{6} = \frac{W_1 x_1 z_2}{6} \quad \ldots\ldots (20)$$

$$B_1 = A_1 + B_1 - A_1 = \frac{W_1 x_1}{24 z_1} [4 z_1 l - 2 z_1^2 - x_1^2] - \frac{W_1 x_1 z_2}{6},$$

or $B_1 = \frac{W_1 x_1}{24 z_1} [4 z_1 l - 2 z_1^2 - x_1^2 - 4 z_2 z_1]$; but $4 z_2 z_1 = 4 l z_1 - 4 z_1^2$,

therefore $$B_1 = \frac{W_1 x_1}{24 z_1} [2 z_1^2 - x_1^2] \quad \ldots\ldots (21)$$

$$C_1 = (r_1 + m) \frac{x_1}{2} - A_1 - B_1, \text{ but } r_1 = \frac{m}{l} (l - x_1) = m - \frac{m x_1}{l},$$

or,

$$C_1 = \frac{m x_1}{2} \left(2 - \frac{x_1}{l}\right) - A_1 - B_1; \text{ but, from Equation (5), } m = \frac{l z_1}{3},$$

therefore, $C_1 = \frac{z_1 x_1}{6} (2 l - x_1) - A_1 - B_1$,

or, $$C_1 = \frac{W_1}{24 z_1} [8 l z_1^2 - 2 z_1^2 x_1 - 4 z_1 l x_1 + x_1^3] \quad \ldots\ldots (22)$$

$$R_1 = \frac{C_1 - B_2}{m} \quad \ldots\ldots (23)$$

$$R_2 = \frac{(A_1 + B_1) + (A_2 + B_2)}{u} \quad \ldots\ldots (24)$$

$$R_3 = \frac{-B_1 + C_2}{g} \quad \ldots\ldots (25)$$

$$(A_2 + B_2) = \frac{W_2 x_2}{24 z_2} [4 z_2 l - 2 z_2^2 - x_2^2] \quad \ldots\ldots (26)$$

$$m = \frac{l z_1}{3} \quad \ldots\ldots (27)$$

$$u = \frac{z_1 z_2}{3} \quad \ldots\ldots (28)$$

$$g = \frac{l z_2}{3} \quad \ldots\ldots (29)$$

$$A_2 = \frac{W_2 x_2 z_1}{6} \quad \ldots\ldots (30)$$

$$B_2 = \frac{W_2 x_2}{24 z_2} [2 z_2^2 - x_2^2] \quad \ldots\ldots (31)$$

$$C_2 = \frac{W_2}{24 z_2} [8 l z_2^2 - 2 z_2^2 x_2 - 4 z_2 l x_2 + x_2^3] \quad \ldots\ldots (32)$$

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The writer has not been able to find any mention of this subject in any of the textbooks in use in this country. Professor H. T. Eddy, in his book, "Researches in Graphical Statics," states the underlying principles of this method very clearly; but he seems to have neglected to carry them out in the manner given in this paper. He gives a graphical discussion of the continuous girder, but the method used does not seem to be so simple as that given here. Professor Burr, in his book, acknowledges his indebtedness to Professor Eddy for the following statements, which the writer thinks are very important, and which he feels sure will some day be responsible for a much simpler method of calculating indeterminate structures, especially arches, than is in use at the present time.

The statements are:

"Let S denote the total shearing stress at any section, P any applied load or external force, M the bending moment at any section, and D the deflection found above, then the six general equations of flexure are:

(1) $$S = \Sigma P;$$

(2) $$M = \Sigma Px;$$

(3) $$P^1 = \frac{M}{EI};$$

(4) $$\Sigma u P^1 = \Sigma \frac{u M}{EI};$$

(5) $$D = \Sigma u P^1 x = \Sigma \frac{u Mx}{EI};$$

(6) $$D_h = \Sigma u P^1 y.$$

"If the beam is originally straight and parallel to the axis of x, u becomes dx, and $y = 0$.

"It is seen from Equations (2) and (5) that $\frac{u M}{EI}$ stands in the same relation to D that P does to M.

"Consequently, if $\frac{u M}{EI}$ be put in place of P, in any graphical construction, D will be represented in the place of M.

"If, therefore, an equilibrium polygon be constructed for any span, by taking $\frac{u M}{EI}$ as loads, instead of P, the vertical ordinates of the polygon will represent the deflections at the sections denoted by the corresponding values of x."

It will be seen by referring to Fig. 1 that if a load be placed at R_2 the triangle $A\ B\ C$ will represent the moments on the beam for this loading, and, therefore, the equilibrium polygon $M\ N\ O$ would now become the deflection polygon.

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The writer was convinced that this was true, some time before he was able to find it stated anywhere. He arrived at his conclusions, however, in a different manner. The general equation of the elastic line is $y = \int\int \frac{M}{EI}$. Now, suppose that the ordinates y represent moments for some system of loading, then $M_1 = \int\int \frac{M}{EI}$.

Then $\frac{d M_1}{d x} = S_1 = \int \frac{M}{EI}$. This would give an equation of the shear of the loading which would produce the deflection curve as an equilibrium polygon. Now, suppose $S_1 = M_2$, that is, suppose this shear curve to be a moment curve for some other system of loading, then $\frac{d M_2}{d x} = S_2 = \frac{M}{EI}$.

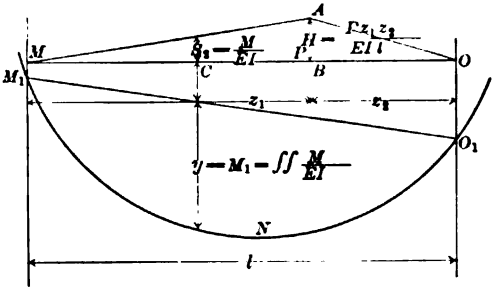

FIG. 10.

Fig. 10 represents graphically the foregoing explanation.

The shear at C for the loading $M\,A\,O = S_1 = \int \frac{M}{EI}$ (the curve representing S_1 is not shown), now suppose S_1 to be a moment curve, M_2, then $\frac{d M_2}{d x} = S_2 = \frac{M}{EI}$. To illustrate further, suppose there is a load P, at B, and that H equals the moment at B from the load P; in other words, $M\,A\,O$ is the moment curve for the load P. Taking Equation (1):

$$y = M_1 = \int\int \frac{M}{EI} = \frac{H x}{6 E I z_1} [2 z_1 l - z_1^2 - x^2],$$

then $$\frac{d y}{d x} = \frac{d M_1}{d x} = S_1 = \int \frac{M}{EI} = \frac{H}{6 E I z_1} [2 z_1 l - z_1^2 - 3 x^2],$$

then $$\frac{d^2 y}{d x^2} = \frac{d M_2}{d x} = S_2 = \frac{M}{EI} = -\frac{H x}{E I z_1},$$

but $$H = \frac{P z_1 z_2}{E I l}, \text{ or } S_2 = \frac{M}{EI} = \frac{P z_2 x}{E I l},$$

or, $$E I S_2 = M = \frac{P z_2 x}{l} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots Q.\ E.\ D.$$

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It may then be stated that, in general, if an equilibrium polygon be drawn with a system of loading represented by the moment curve for any particular loading, the equilibrium polygon becomes the deflection curve for that particular loading. The writer wishes to call attention to the fact that this is true only when the whole system is in equilibrium. As, for example, in a cantilever beam there must be introduced a moment at A (Fig. 11), or an external force at B to keep in equilibrium the loading represented by the influence line for the load P.

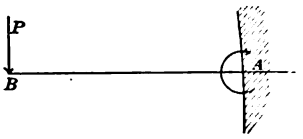

FIG. 11.

In this paper the writer has given much which is probably already well known to most engineers, but he hopes that some new ideas may be suggested which will lead to more advanced work in this line.

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DISCUSSION.

C. H. LINDENBERGER, Assoc. Am. Soc. C. E. (by letter).—The inventor, or the graphical specialist, is usually a poor judge of the merits of a new graphical method, because the difficulties that lie in the way of its assimilation by the busy computer, whose time is precious, do not exist for him. This is the writer's apology for making a brief summary of a graphical method perfected by him ten years ago. Nothing on the subject has since been published by the writer. The original articles* dwelt only upon the most general case, and many parts are difficult to understand on account of that fact. This brief summary is restricted to a girder resting on four supports, with equal end-arms and constant moment of inertia. It is a method of constructing the actual equilibrium polygon for the actual loads, so that reactions, shearing stresses, and moment ordinates can be at once scaled from the drawing. No quantity needs to be computed analytically; but well-known methods of computing certain factors graphically are omitted, for obvious reasons. Mr. Lindenberger.

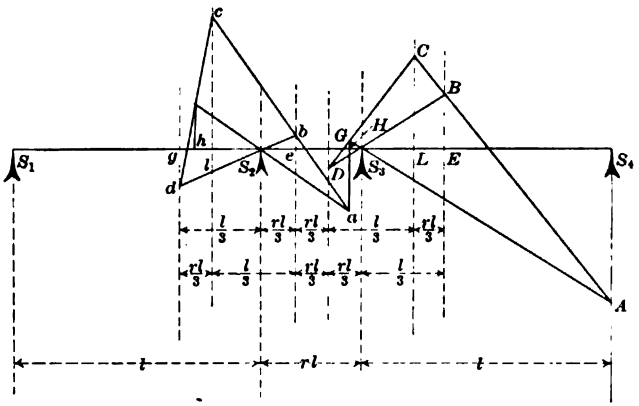

FIG. 12.

P is a load on the end-span or arm;
l is the length of each of the two equal arms;
$r\,l$ is the length of the middle span or drum panel;
$k\,l$ is the distance from the load to the end of the bridge;
R_1, R_2, etc., are the reactions at the supports S_1, S_2, etc.;
H is the pole distance of the force polygon;
n is an arbitrary multiplier to be explained later; its value is a mere matter of convenience;
Σ is the sign of summation.

The first proceeding is to find the "fixed points," as in Fig. 12.

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Mr. Lindenberger.

S_4, A and $H\,a$ are taken of any convenient length, and $A\,C$ and $a\,c$ may have any convenient slope. The intersections, with the verticals drawn through the points E, L, S_3, H, G, e, S_2, l, and g, determine the remainder of the construction. Notice that $E\,L = S_3\,G = e\,S_2 = l\,g = \frac{r\,l}{3}$, and that $L\,G = E\,S_3 = e\,l = S_4\,g = \frac{}{3}$.

The fixed points are at H and h. It may be proved that

$$S_3\,H = \frac{r^2\,l}{2+3\,r};\quad S_2\,h = \frac{2\,(1+r)\,l}{3\,r^2+10\,r+6}.$$

After having thus found these points, mark them plainly and draw verticals through them and erase all other lines except the line which represents the drawbridge (S_1, S_4) and the verticals through the points of support. Then proceed to the construction of Fig. 13. In this diagram the shaded portion is the actual equilibrium polygon for the

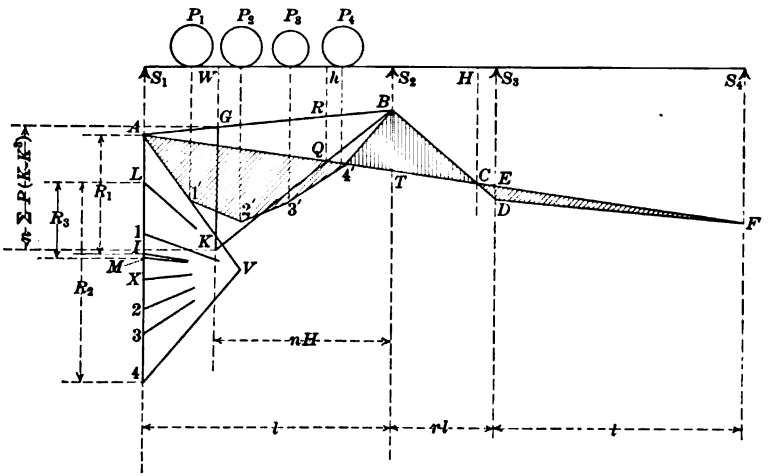

FIG. 13.

drawbridge as a continuous girder for the actual loads. The area shaded diagonally indicates compression on the upper chord; that shaded vertically, the opposite, or tension. To construct it, draw the load line, $A\,1$ = first load; $1\,2$ = second load, etc., and construct the equilibrium polygon, $A\,1'\,2'\,3'\,4'\,B$, as if the end-arm or span rested simply on its end supports and was disconnected from the remainder of the bridge. For the purpose of obtaining accurate intersections and directions it is necessary to lay off $\Sigma\,P\,(k-k^3)$ to a larger scale than is convenient for the load line. Hence, lay of $S_2\;W = n$ times the pole distance ($= n\,H$) and draw the vertical $W\,K$. On this vertical lay off $G\,K = n\,\Sigma\,P\,(k-k^3)$. There are several methods of computing this quantity graphically, and all the valuable ones are well known to

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those familiar with the continuous girder. Draw $B\,K$. It intersects the vertical through h at Q. Draw $A\,Q$, and produce it to F. Then $A\,F$ is the closing line of the equilibrium polygon for the bridge as a continuous girder. It cuts the vertical through H at C. Draw $B\,C$, and produce it to D on the support vertical, and draw $D\,F$. The equilibrium polygon is now complete. The reactions are found by drawing lines from the pole parallel to the sides of the new equilibrium polygon. Thus, $I\,A = R_1$; $4\,L = R_2$; $L\,M = R_3$; $M\,I = R_4$. The shearing stresses and moment ordinates can now be scaled directly from the drawing. Mr. Lindenberger.

The proof for the construction can be found in the articles referred to, contributed to the *Journal* of the Franklin Institute. The method of treatment of a pivotal bridge will be evident from the foregoing, as only one fixed point need be found, where the arms are equal.

H. A. LaChicotte, M. Am. Soc. C. E. (by letter).—The graphical method of finding the reactions of a continuous girder, as presented by the author, is a valuable contribution to the literature on swing bridges. The simplicity of the method and its ease of application will commend it to American engineers. The author has presented the subject so clearly and fully that there seems to be little of interest to be added. Mr. LaChicotte.

The formulas expressing the Theorem of Three Moments, as applied to swing bridges, in American practice, are based on two assumptions which no doubt have impressed engineers and students as radical and unwarranted. The first of these is that moments of inertia are assumed as constant, which actually vary through wide limits—in some cases as much as 400 or 500 per cent. The second is that all points of support are at the same level, or on the same straight line. It is true that with proper care in construction the supports may be brought to the same level, but the danger of unequal settlement, with the consequent disturbance in the stresses of the structure, usually gives rise to feelings of apprehension. It may be shown, however, that no serious error is likely to result by reason of the foregoing assumptions.

Let us consider a beam of constant moment of inertia, resting on three points of support, with two equal spans uniformly loaded throughout:

l = total distance between end supports;

p = unit load per foot.

If the beam were absolutely inflexible, the problem of finding the reactions would be incapable of solution; for the least settlement of one of the supports would relieve it of load entirely, and there would result either a simple span with end supports, or two cantilever spans balanced on the middle pier.

It is owing to the flexible nature of the beam—to the fact rather than degree of flexibility—that the reactions and stresses are determinate.

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Mr. LaChicotte. If, in Fig. 14, the end supports be removed, the girder will act as a double cantilever, and the maximum deflection will be expressed by the formula

$$\delta = \frac{3\,p\,l^4}{384\,E\,I}.$$

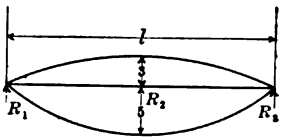

FIG. 14.

If the middle support be removed, the girder will act as a simple span, and the center deflection will be expressed by the formula

$$\delta = \frac{5\,p\,l^4}{384\,E\,I}.$$

It will be noticed that these deflections bear to each other the ratio of 3 to 5, and vary directly as the load and inversely as the moment of inertia.

In the case of the simple span, if a single force, R_2, directed upward, be applied to the center of the beam, so that the downward deflection is just overcome, and the beam rests on three supports at the same level, the value of R_2 may be found by equating its deflection to that produced by the load. The deflection caused by a single load, R_2, applied at the center of the span, is $\delta = \frac{R_2\,l^3}{48\,E\,I}$, and, placing this equal to the deflection due to uniform load, we have

$$\frac{R_2\,l^3}{48\,E\,I} = \frac{5\,p\,l^4}{384\,E\,I}.$$

Whence, $R_2 = \frac{5}{8}\,p\,l$, an expression no longer involving the moment of inertia.

While the flexibility of the beam is a prerequisite in the determination of the middle reaction, the amount of that reaction is independent of the degree of flexibility so long as the supports are all at the same level.

It was found previously that if the middle support be removed, so that the beam acts as a simple span, the downward deflection is 5 units, while, if the middle support be raised sufficiently to annul the end reactions, the reaction at the middle support would produce an upward deflection of 3 units, making a total range of 8 units. It is also shown above that it requires a center reaction equal to five-eighths of the total load to overcome five-eighths of the total deflection and bring the supports to the same level. The reaction at the center varies directly according to the position of the center support with reference to the other two. If, for instance, the center support settles one unit below the others, it gives up one-fifth of its reaction, and, under total uniform load, the two arms act substantially as two simple spans.

The unit above mentioned has the value $\frac{p\,l^4}{384\,E\,I}$, which may be

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readily computed for a certain span of known moment of inertia. It equals 1 in. for an ordinary swing bridge of approximately 300 ft. span, deflecting under dead load, and without end supports. Mr. LaChicotte.

It matters not whether the difference in level of supports results from unequal settlement of the piers, or from improper lifting of the ends of the span when closed; and, when neither of these conditions exists, their equivalent is produced by the unequal expansion of the chords of the bridge, where the upper chord is exposed to the sun while the lower one may be more or less in the shade.

Every engineer who has had to do with the actual operation of swing bridges knows how difficult it is to realize and maintain anything like theoretical conditions; and, consequently, over-refinement in methods of computing stresses is wholly unnecessary. The only safe rule is to assume conditions of such wide range as to surely cover any probable variations in the reactions. The knowledge of these uncertainties will be enough to commend the author's method, even to those engineers who are prejudiced against graphical methods in general, and especially where results are dependent upon the accuracy with which curves can be plotted. In the present case, a single diagram being required for all spans with equal arms, for each number of supports, the deflection curves may be drawn, once for all, with all necessary care and accuracy. The results will probably conform more closely to the assumed conditions than the latter do to the conditions actually existing.

It is not claimed here that accurate results may not be obtained by the graphical method, but there is, among engineers, a well-known prejudice to its use in final computations, which, no doubt, arose from unfamiliarity with the subject, or from unsatisfactory results obtained through lack of proper care in the construction of the stress diagrams. Engineers, however, who make frequent use of the graphical method are better able to appreciate both its usefulness and its limitations.

It is in its application to the continuous girder that the graphical method has received its highest development, and where its simplicity and directness render it in many respects preferable to the analytical method.

As regards the derivation of the elastic curve from the curve of moments, the following may be helpful in reaching a clear understanding of the method.

The equation of moments of a uniformly loaded beam is $M = \frac{p\,x}{2}(l - x)$, of which the second x-derivative, $\frac{d^2 M}{d\,x^2} = -\,p$, is a constant, and expresses the constant or uniform load per unit of beam. If, however, the unit load is not constant, but is a function of x, it is only necessary to express the load at any point of the beam in terms of x, and substitute this value for the unit load, p, in the foregoing expression.

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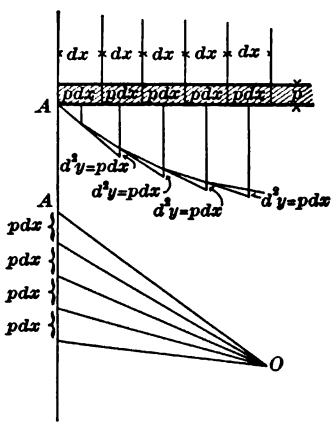
Mr. LaChicotte. The resulting value of M, after two integrations, would give the correct value of the moment at any point of the beam. For instance, if the load be assumed to vary uniformly, from 0 at one end of the beam to p at the other, the unit load at any other point would be $\frac{p\,x}{l}$. The second differential becomes $\frac{d^2\,M}{d\,x^2} = -\frac{p\,x}{l}$, and, performing two integrations, there results for the moment at any point:

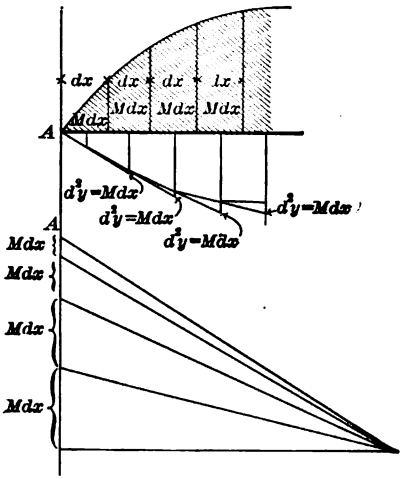
$$M = \frac{p\,x}{6\,l}\,(l^2 - x^2).$$

In like manner, the load may be assumed to vary as a parabola, and if M, a function of x, represents the unit load at any point of the span, then, if y be the ordinate of the moment curve, $\frac{d^2\,y}{d\,x^2} = -M$.



Pole distance Arbitrary
Assumed=Unity
pdx=Constant

FIG. 15.



Pole distance=EI
Mdx=Variable

FIG. 16.

Before integration, M must be expressed in terms of x. If, in addition, M be divided by $E\,I$, which amounts to changing the scale by which the ordinates of the moment curve are plotted, or, in other words, fixing the pole distance of the polygon of forces, there results $\frac{d^2\,y}{dx^2} = -\frac{M}{E\,I}$, the double integral of which may be the equation of the moment curve for a load $\frac{M}{E\,I}$, or the deflection curve for the unit load which produces the moments, M.

Figs. 15 and 16 show the similarity of the moment and deflection curves so clearly that no explanation is needed. It is only necessary

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to bear in mind that if p be the load per unit of length of beam, then $p\,dx$ is the load on an infinitely short length, dx. Mr. LaChicotte.

From similar triangles, in the diagrams of moments and force polygons, there may be written $\frac{d^2y}{dx} = -\frac{p\,dx}{1}$, and $\frac{d^2y}{dx} = -\frac{M\,dx}{EI}$, the second member of each equation being taken minus, because, the ordinates y, being measured to the concave side of the curves, the second derivatives, $\frac{d^2y}{dx^2}$, are essentially negative.

GEORGE F. BARTON, Esq. (by letter).—Mr. Lindenberger gives another method by means of which the moments and shears in the continuous girder may be found graphically. The writer has not been able to obtain copies of the *Journal* of the Franklin Institute, referred to in explanation of this method, and, therefore, could not follow it throughout. The method seems to be very complete, but every change in the position of the loading, or the style of the loading, necessarily changes his diagrams. It was for this reason that the writer did not go any farther in his own method. Mr. Barton.

In the method described in the paper it is only necessary to lay out one diagram once for all spans of equal arms, no matter what kind of loading may be on the structure, or what length the arms may be. For complicated structures, like draw-bridges and arches, most designers would be glad to be able to find the reactions by a simple method, and would not care to get the moments and shears, if, by doing so, they were compelled to do more work than would be required to compute them analytically, after having found the reactions graphically by a simple method. This is the writer's excuse for giving a solution, which, as a graphical solution, is incomplete.

Mr. La Chicotte recognizes the value of this method, from this point of view, and shows that, while the ordinary assumptions may be very far out of the way, engineers seem to be afraid to use a graphical method which will save them considerable time and worry, and yet which will, in all probability, give results which are as close to the actual stresses as if there were a good deal of refinement in the calculations.

Mr. La Chicotte's very simple explanation of the deflections of beams under different loadings is very interesting, and shows clearly how a little settlement of the supports, or a change in the temperature of the chords, may change the stresses considerably.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 918.

NOTE ON THE CONSOLIDATION OF MORTAR.

By GEORGE W. RAFTER, M. Am. Soc. C. E.

The writer, in his paper, "On the Theory of Concrete,"* has cited data showing the consolidation of concrete under ramming, and recently, a correspondent has called his attention to the fact that, aside from a single statement by Professor Ira O. Baker, on page 89 of the Ninth Edition of his "Treatise on Masonry Construction," there is no information in American engineering literature as to the consolidation of mortar under ramming.

In looking over his note books, the writer has found a record of the tests given in the accompanying table, and these throw some light on this subject.

The cements used were the same as in the concrete tests, referred to in the paper "On the Theory of Concrete." Details as to tensile strength, etc., may be found in the writer's "Report on Concrete."†

The table herewith presented is so self-explanatory that it remains merely to point out the reason for the variations in the percentage of consolidation under ramming, which, for dry mortar made from Empire, Iron-Clad and Champion cements, ranges from 0.9 to 10.9%, as shown in Column 9.

* *Transactions*, Am. Soc. C. E., Vol. xlii, p. 104.

† In the Report of the State Engineer and Surveyor of New York, for 1897.

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The writer's view is that the amount of consolidation under ramming is governed chiefly by the thoroughness of the mixing. When, as in the case of Iron-Clad 1 to 2 mortar, in the accompanying table, there is a consolidation of 10.9%, probably nothing is to be said except that the mixing was indifferently done.

The plastic mortars show the effect of greater lubrication in mixing, and, consequently, less consolidation under ramming.

The practical value of these data, in enabling one to approximate more closely the quantity of mortar ingredients required for a given volume of masonry, is so obvious that it does not require discussion. It may be stated, however, that this series of tests is hardly extensive enough to fully settle the general question of consolidation of mortar. For the present, therefore, they are cited as rough guide boards; with the hope that someone may be able to carry out a more extended series.

The cost of making such tests, when the details are thoroughly thought out beforehand, is not so great that it should deter an engineer in charge of works of any magnitude. In many cases, a small amount of money, judiciously expended in studying technical conditions, will result in better work for a given expenditure.

EXPERIMENTS ON CONSOLIDATION OF MORTAR.

Consistency of mortar.	Brand of cement.	Proportions of cement and sand.	Sand, in cubic feet.	Cement, in cubic feet.	Pounds of water per cubic foot of mortar.	Quantity of unrammed mortar, in cubic feet.	Quantity of rammed mortar, in cubic feet.	Percentage of consolidation under ramming.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Dry.....	Empire.	1 to 2	4½	2¼	16.6	5.62	5.38	4.4
"	"	1 to 3	4½	1½	15.8	4.86	4.82	0.9
"	"	1 to 4	5	1¼	16.5	5.32	5.06	4.7
"	Iron-Clad.	1 to 2	4½	2¼	16.3	5.79	5.32	10.9
"	"	1 to 3	4½	1½	16.1	4.96	4.63	6.5
"	"	1 to 4	5	1¼	15.5	5.28	4.89	6.9
"	Champion.	1 to 1	3	3	17.4	4.81	4.55	5.7
"	"	1 to 2	4½	2¼	16.2	5.85	5.11	4.7
"	"	1 to 3	4½	1½	16.2	4.79	4.60	4.1
							Mean.....	5.4
Plastic.....	Empire.	1 to 2	4½	2¼	19.0	5.80	5.68	3.4
"	"	1 to 3	4½	1½	17.7	5.06	4.92	3.5
"	"	1 to 4	5	1¼	16.7	5.42	5.16	5.2
							Mean.....	4.0
						Mean of all.....		5.1

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 919.

SOME DEVICES FOR INCREASING THE ACCURACY OR RAPIDITY OF SURVEYING OPERATIONS.*

By Walter Loring Webb, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Gerard H. Matthes, C. A. Sundstrom, John F. Hayford, Willard D. Lockwood, L. C. Sabin, Oscar Erlandsen, Horace Andrews, George A. Taber, Leonard S. Smith and Walter Loring Webb.

The devices described in this paper were all designed and constructed under the direction of the writer. All have been subjected to practical use, and their merits demonstrated. In some cases the general principle was adapted from a similar device on a very different instrument. In one instance only (so far as known) has any one of the devices been illustrated as an instrumental feature by an instrument maker, and, even in that case, not until long after its construction and use by the writer. It is confidently expected, however, that, as is usual in such cases, there are many who may claim priority of invention on some of these devices. Regardless of the validity of such claims, the writer maintains that the devices are not in such general use as their undoubted merit will justify, and he believes that he is doing a service to the profession by calling attention to them.

* Presented at the meeting of February 19th, 1902.

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I. Vertical Arc on Plane-Table Alidade.—The disturbances of a plane table, due to its being used as a drawing table, can readily be kept within limits which will not essentially vitiate the accuracy of those operations which are peculiar to the plane table. But the advantages of using the stadia method in connection with the plane table are great. To obtain differences of elevation with the accuracy generally required necessitates the determination of the vertical angle to the nearest minute of arc. No matter how carefully the plane table was originally leveled up, a brief use will put it out of level, not only 1 minute of arc, but possibly 5 or 10 minutes. Re-leveling will inevitably turn the table in azimuth enough to vitiate future angular work. The device illustrated in Fig. 1 of Plate I has been tried, with very gratifying results. The screw, *A*, moves the vernier arc, which can revolve

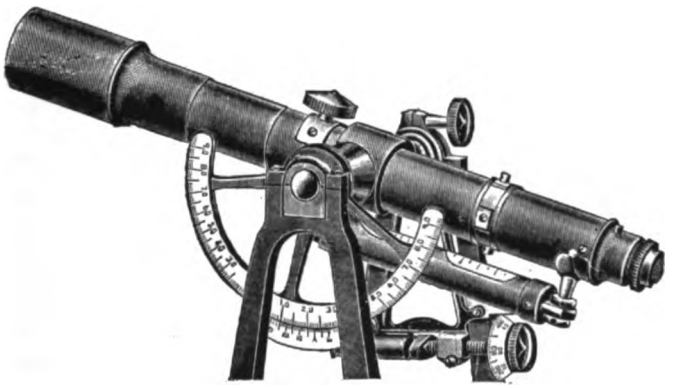

FIG. 1.

around the telescope axis. The graduated arc is rigidly attached to the telescope axis. On the vernier arc is a level bubble, adjusted so that when the bubble is in the center and the line of collimation of the telescope is actually horizontal, the vernier reading is 0 degrees. Therefore, for any pointing of the telescope, up or down, even when the table is slightly out of level, it is only necessary to adjust the bubble to the center, and the vernier reading is the true vertical angle. The principle was suggested to the writer by the vertical arc and bubble attached to a zenith telescope. The amplitude of motion of the vernier arc need not be great, for if the table becomes very much out of level, it indicates a disturbance which would necessitate re-setting for azimuth as well as re-leveling.

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II. Vertical Arc on Transit.—Few transits with large vertical arcs have passed through much active field service without the arcs becoming more or less bent. An inspection of Fig. 1, taken from the catalogue of a well-known maker, and which exhibits the usual type of semi-circular vertical arc, shows the reason. No matter how the telescope is set while the instrument is carried, the ends of the arc protrude, and, like a sore finger, are sure to strike something (merely a twig of a tree, perhaps, or the side of the instrument-box when the instrument is being put in or taken out) which will bend it slightly. To reduce this danger, the arc is usually made of small radius, so small that, if graduated nominally to single minutes, even the determination of the nearest minute becomes uncertain, and the estimation of half minutes becomes impracticable. A comparison of Fig. 1 with Fig. 2, of Plate I, will show how the danger of striking and bending the arc has been reduced in the design of the latter. Here there are two verniers attached to a plate which is screwed to the standard. The zeros of the verniers are precisely 60° apart. The graduated arc, attached rigidly to the telescope axis, is numbered in two rows from 0° to 60° each way, and is graduated for a few degrees over 60°, to permit vernier readings of angles which are nearly 60 degrees.

When the telescope is pointing up (Fig. 3, of Plate I), the right-hand vernier is read, and this will accommodate any angle up to the practicable limit of use (without a prismatic eye-piece). For an angle of depression, the left-hand vernier will be used, and this will accommodate any angle down to where the vision is obstructed by the plate. This device enables a long radius to be used, as is shown graphically in Fig. 4, of Plate I. The radius in that case is 3½ ins. A 7-in. circle permits not only easy certainty in reading graduations to 1′, but even to 30 or 20 seconds. And yet a 7-in. semi-circle of the plan of Fig. 1 would almost inevitably be struck and bent during the course of a very short service. Of course, the instrument would never be carried with the arc inverted, as in Fig. 4, of Plate I.

III. Combination of Double Vernier and Adjustable Vernier.—When it is desired to take a large number of stadia "shots" from one station with a transit, the time required is so great that the transit will generally settle somewhat out of level. Re-leveling will almost inevitably shift the transit in azimuth. In addition, the plate bubbles

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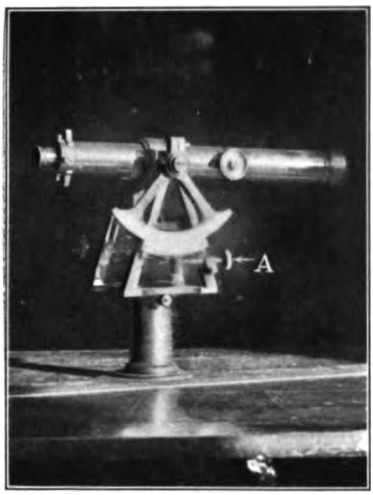

FIG. 1.—VERTICAL ARC ON PLANE-TABLE ALIDADE.

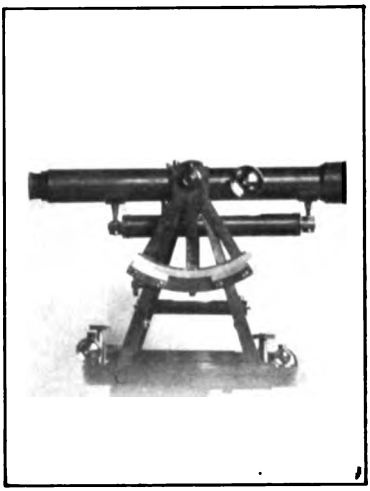

FIG. 2.—SHORT VERTICAL ARC ON TRANSIT.

FIG. 3.—SHORT VERTICAL ARC ON TRANSIT.

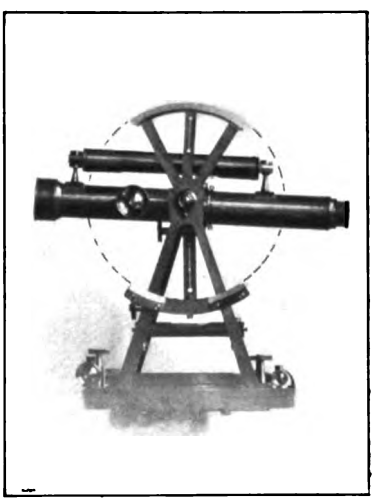

FIG. 4.—SHORT VERTICAL ARC, SHOWING LONG RADIUS.

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of a transit are usually too coarse to serve as the basis of the accurate measurement of a vertical angle to the nearest minute or to 30 seconds. Therefore the adjustable vernier principle was tried. In Fig. 1, Plate II, the two verniers are attached to an arc swinging around the telescope axis but held in position by the screw, *A*. On this arc is placed a bubble, *B*, which is much more sensitive than the usual plate bubbles, and which will readily indicate a change of much less than 1′ of arc. As with the plane-table arc, the vernier need not have a large amplitude of motion. The sensitive vernier level is a "watch level." It shows the steadiness of the instrument, and needs only to be adjusted when the bubble shows it to be necessary. The actual use of this bubble on a transit was, to the writer, a revelation of the motion to which a transit is subject, even under ordinary conditions, a motion which is practically unnoticeable with the coarser plate bubbles. It is found necessary to readjust the vernier quite frequently, and, knowing the angular value of the motion of this vernier bubble, the actual uncertainty of usual vertical arc determinations becomes very apparent. A circuit of several miles was recently run with an instrument equipped with this device, and the resulting error of closure of levels was far within what could have been expected with the usual instruments. This device was applied to a transit, under the direction of the writer, by Heller and Brightly, of Philadelphia.

IV. Stadia Slide Rule.—Stadia plottings on a plane table require that stadia "reductions" be made in the field. It has similarly been found that sketch-board plottings in the field, of difficult topography, taken with a transit and stadia, reduce the chances of errors of plotting, and this likewise requires the "reductions" to be made in the field. These reductions are made: (*a*) by numerical computation, with the aid of tables—which is so laborious as to discourage the use of the stadia method; (*b*) from a chart which, if large enough for accurate work, is too large and clumsy for field use and is inconvenient even for office use; and (*c*) by stadia slide rules. The slide rules hitherto on the market are either so small (the logarithmic unit being so short) that the required quantities are not obtained with sufficient accuracy, or else they are so large that they are clumsy. By adopting the cylindrical principle, of which the Thacher rule is a familiar example, a slide rule with a logarithmic unit 12½ ins. long has been made, and yet the total

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length of the instrument is but an inch or so greater. This length of logarithmic unit was chosen after considerable study of the accuracy required for ordinary conditions. It is ample, and yet the instrument is so small as to be readily carried and used in the field. This device has just been manufactured for the writer by Keuffel and Esser, of New York. The true horizontal distance is obtained by computing the correction to the nominal distance. The value of this correction is obtained with much greater ease and accuracy than that required to read off directly from the scale the true horizontal distance, for the changes in $\sin.^2$ of small angles are much more rapid than the changes in $\cos.^2$ of the same angles.

Engineers unfamiliar with such an instrument will be surprised at the rapidity with which two men (one calling off and recording, and the other using the slide rule) will dispose of a series of several hundred stadia shots. Inexperienced men can be taught to use the rule effectively in a few minutes. Such an instrument removes what has been a barrier to the adoption of stadia methods.

V. Tension Frames for Steel-Tape Measurements.—The development of methods of steel-tape measurements has resulted in work being done with an accuracy of 1:1 000 000. If a tape has a cross-section of 0.00203 sq. in., and a modulus of elasticity of 30 165 000 lbs. per square inch,* a variation in tension of slightly less than 1 oz. will change its length by one-millionth part of its length. Extreme accuracy in tape measurements, therefore, can only be obtained when the actual tension may be determined with an accuracy which is beyond question. An ordinary spring balance, the closest reading of which is ¼ lb., is, of course, useless for such refined work. Even if the balance reads to ounces, its reading when held horizontally is very different from its reading when held vertically, and an allowance must be made for this. And, after all, there is always the feeling of uncertainty of the measurement of a force by a spring which may vary, rather than by the known force of gravity. J. B. Johnson, M. Am. Soc. C. E., describes† a somewhat rude contrivance for measuring tape tension by the moment of a weight about a knife-edge.

The fundamental idea has been elaborated into the device illustrated in Fig. 2 of Plate II, and Figs. 1 and 2 of Plate III. In Fig. 2, Plate II, is shown, separately, the tension apparatus. The tape is

* Actual figures for the Univ. Penn. 300-ft. tape.

† "Theory and Practice of Surveying."

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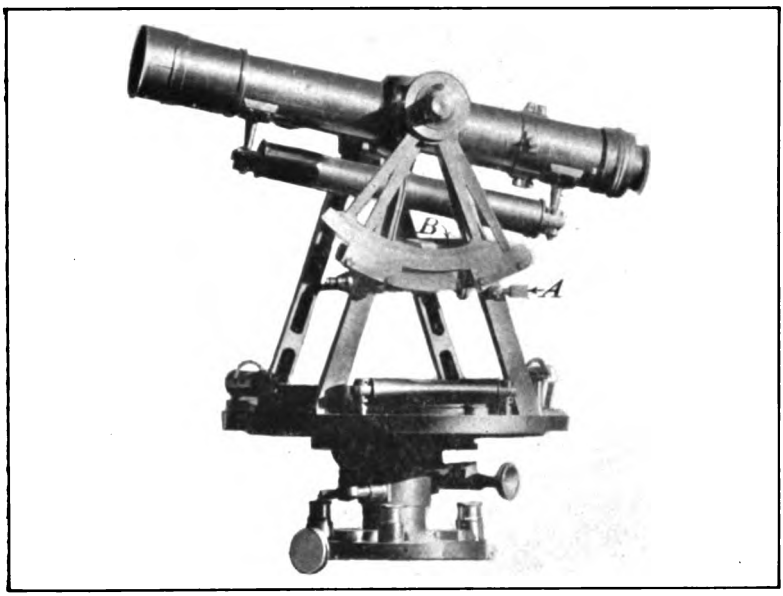
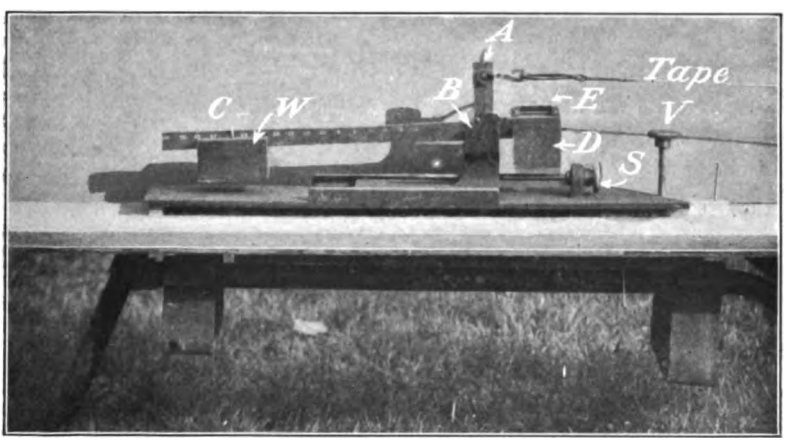


FIG. 1.—DOUBLE VERNIER AND ADJUSTABLE VERNIER.



NO. 2.—TENSION FRAME FOR STEEL TAPE.

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attached to a forked hook, which plays on knife-edges at *A*. The line joining the knife-edges, *A* and *B*, is perpendicular to *B C*. Therefore, when *B C* is horizontal (as shown by the level bubble at *E*), *A B* is vertical, and the level tape is pulling with a moment about *B* which is exactly determinable. It was, of course, a simple matter to design the counterweight, *D*, the sliding weight, *W*, and the various dimensions so that the various notches on the scale bar would correspond, with even greater accuracy than can be utilized, to the tension actually existing in the tape. In this apparatus, after the accuracy of design and adjustment of the counterweight and bubble is once satisfactorily determined, no doubt can exist as to the accuracy of its determination of tension.

Of course, certain other details are necessary for practical use. The screw, *S*, moves the standard supporting the fulcrum forward or back in grooves, as may be necessary to produce the required tension. The screw, *V*, raises or lowers the bed-plate so that the tape between *A* and the top of the stake (Fig. 1, Plate III) may be truly horizontal. A projecting pin on the lower side of the bed-plate passes through a transverse slot in the supporting plate, *P*. This slot enables the tension apparatus to be adjusted laterally to proper alignment, and yet holds it in position longitudinally. The screw, *V* (see Fig. 2, Plate II, unfortunately hidden in Fig 1, Plate III) rests on the supporting plate, *Q*. The frame, *Z Y Y*, is made of 1-in. pipe. Only an approximate adjustment for position and height is necessary when setting the frame on the ground, for the tension apparatus may be adjusted laterally, longitudinally and vertically by the means described above.

At the other end of the tape a somewhat similar frame is used. A wing-nut, *X*, turns on a screw and pulls the tape until the zero mark is precisely where desired. The piece holding the screw likewise has a lateral motion, by means of pins projecting through a transverse slot in the plate, *R*, which supports it. No vertical motion is allowed for, as it is unnecessary, it being really preferable that the hook should be somewhat lower than the stake, so that the tape should bear on the stake slightly. The pieces, *Z* and Z^1 (Figs. 1 and 2, Plate III), are short pipes pointed at their lower ends and at their upper ends terminating in tees which will turn loosely on the horizontal pipes which form the ends of the frames. By varying the angle of this pipe, Z^1, or by forcing it deeper into the ground, the height of the back end of the frame can readily be adjusted with sufficient accuracy. The pipes, Y^1 Y^1, are also

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pointed, and take up the pull of the tape on the whole frame. Of course, this frame (Fig. 2, Plate III) should be used at the rear end of the tape. A series of hooks on the left end of the screw-bar, on one of which the tape ring may be hung, obviates the necessity for excessive motion of the wing-nut.

As the tape, when at the required tension, is held mechanically, and not by means of muscles which may be tired and nervous, the positions of the end marks on the hubs may be determined leisurely, and with as great precision as desired.

The ability to produce and measure exactly any desired tension in the tape permits a very exact determination of the modulus of elasticity, assuming that a linear vernier or micrometer is also available. It is simply necessary to note the elongation of the tape between tensions of 8, 10, 12, 14 and 16 lbs. A large number of independent solutions is thus obtainable, the agreement of which will furnish a criterion of their reliability.

This apparatus was made, several years ago, from designs by the writer, for the Civil Engineering Department of the University of Pennsylvania, by Tinius Olsen, of Philadelphia.

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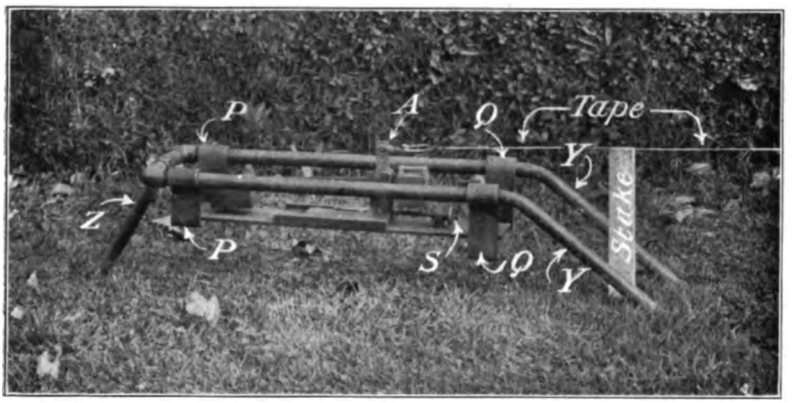

FIG. 1.—FRONT TENSION FRAME FOR STEEL-TAPE MEASUREMENTS.

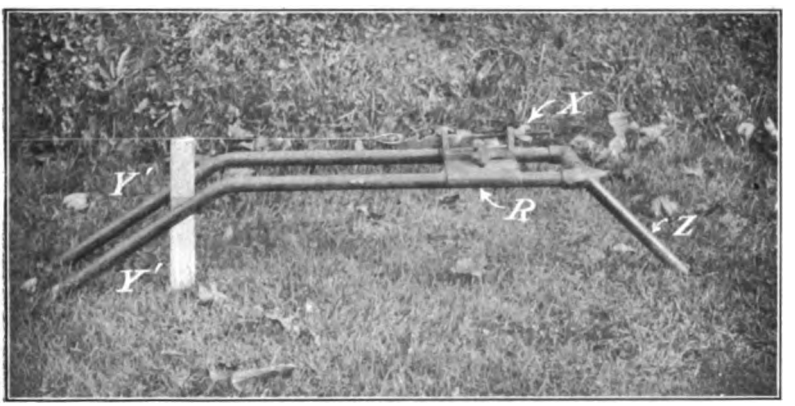

FIG. 2.—REAR TENSION FRAME FOR STEEL-TAPE MEASUREMENTS.

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DISCUSSION.

GERARD H. MATTHES, Assoc. M. Am. Soc. C. E. (by letter).—Professor Webb's efforts are a step in the right direction, and are characteristic of the tendency that has manifested itself of late years to simplify instruments and methods of surveying in order to lighten and dispatch work, and, at the same time, maintain a standard of precision sufficient for the purposes in hand. During the last two decades an enormous advance has been made in the construction of surveying instruments. Mr. Matthes.

In the year 1870 the Great Trigonometric Survey of British India was still laboring with 36-in. theodolites weighing 370 lbs. when mounted, and as much as 1 050 lbs. when packed for shipment, inclusive of accessories. The present generation of surveyors may well consider themselves fortunate to be able to look back upon these ponderous machines as interesting relics of days gone by. Thanks to the modern dividing engine and to improved machine tools, it is now possible to turn out theodolites with smaller plates graduated more accurately than the old hand-divided 36-in. circles.

In this country, the United States Coast and Geodetic Survey is using 12-in. theodolites for work of the highest order, while the United States Geological Survey finds that 8-in. theodolites answer its requirements. In mine surveying and in other fields, such as base-measuring and plane-table work, similar wonderful advances have been made, simpler and better forms of instruments taking the place of the old, complicated and cumbersome types.

Much remains to be accomplished, however. Many surveying instruments are yet far from being all that is desired, as regards fitness for the work for which they are intended. Their development is a process of evolution. The demand at the present day is not so much for the invention of new forms of instruments as for the perfection of those now in use. It is difficult to design any instrument of precision, with a complication of parts, and capable of delicate adjustments, that shall, at the same time, be easily manipulated, compact, readily transported, and strong enough to withstand the strains imposed upon it by rough handling and by sudden variations of temperature and humidity, which will not be thrown out of adjustment continually. Yet this is precisely what is desired of surveying instruments. Only through long use and experience can the best types be evolved.

Having had experience with instruments of different kinds, under a variety of conditions, from the intense heat of southern Arizona to the cold temperatures of northern Montana, the writer has been in a position to gain a full knowledge of the weak points in modern surveying instruments and the difficulties to be overcome in remedying them.

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Mr. Matthes. It is therefore with joy that he hails the improvements suggested by Professor Webb, albeit some of his devices are capable of further improvement.

The shortening of the vertical arc on transits by the addition of a vernier seems to be an excellent expedient. The change is a decided benefit, while the extra vernier is in no sense a burdensome feature, and would tend to obviate errors in reading plus or minus angles, provided, of course, that the numbering on the arc and the verniers be made so as to easily distinguish plus angles from minus angles. This can be accomplished by slanting the numbers, as is frequently done on transit plates which are numbered both ways. The question arises, however: If, according to Professor Webb's views, an arc of 60° amplitude is safe as regards injury, why not cut the ordinary form of arc down to that limit, thus leaving 30° on each side of the zero mark, and retaining only one vernier? Excepting instruments used in solar observations, where the long arc is a necessity, an arc of this amplitude would answer for all ordinary surveying operations. In the steepest hillside work stadia angles never exceed 30°, and anyone who takes stadia angles exceeding 25° is doing an inferior quality of work, at best, unless the distances are very short. The writer, therefore, would suggest that for transits not provided with solar attachments, or intended for special lines of work entailing the measurement of large vertical angles, an arc of 60° amplitude, graduated after the common pattern, is fully sufficient, and that, therefore, the author's device seems to be valuable for only such instruments as require long arcs.

Professor Webb's adjustable vernier attachment is a useful addition, particularly as regards its application to tachymeters. Obviously, the addition of an extra spirit level is no burden to a transit. In the case of the alidade, however, it must be thus regarded. It should be borne in mind that the plane table and alidade, considered as one instrument, is essentially used for mapping and reconnaissance purposes. This implies that it must be especially adapted to work in regions away from centers of civilization, where means of transportation are limited and rough. In other words, the table as well as the alidade should be made durable and strong, with as few parts requiring adjustment as possible, and yet, withal, be compact and light. In such an instrument every delicate part is a constant source of worry to the topographer during the journey or the march, and those who can speak from experience will not welcome additional level vials.

In the work of the United States Geological Survey, particularly in the Western States, the hard usage to which alidades are put is worthy of more than passing comment. In open terrain the topographer moves from point to point in a buckboard, and the jolting of these vehicles over prairies of bunchgrass pitted with badger and

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prairie-dog holes, or over a sagebrush country, can be understood only by those who have had the experience. In rugged country, recourse must be had to saddle and pack horses, and for hours the topographer may be required to trot over narrow trails, through timber and brush, while the pack animal behind him scrapes his precious burden through the best he can, the boxes or tripod frequently receiving hard knocks against tree trunks or projecting ledges; knocks which are unavoidable, and compared with which the jolting of the load due to the animal's gait is not worth considering. In addition, the topographer may have to ford mountain torrents time and again during the course of a day, and take chances of wetting his outfit. In ascending high peaks, even the faithful pack animal must be dispensed with, the topographer and his assistant dividing the instrument outfit between them, and carrying it up the mountain on their backs. Starting at 6 A. M., for instance, they may reach the top at 10 A. M. About four hours remain in which to do all the work, if another ascension of the peak is to be avoided. As a rule, the work averages from 40 to 80 observations or more, and must be done rapidly, if any time be left for luncheon and for sketching. Consequently, it is important that the alidade should come out of its box none the worse for the rough usage it has received in transportation. and its adjustments must be few and rapidly disposed of, so that in a few moments it will be capable of doing first-class work. On the way down the mountain the topographer, carrying the alidade box on his back, may have to jump down a dozen or more ledges 5 or 6 ft. high, and similar jumps may have to be made in working through "down timber." Mr. Matthes.

The foregoing is no exaggeration of facts. Scores of men are annually engaged on surveys where that is the daily routine, and the alidade must be proof against rough handling, or it is not fit for the work.

Regarding the form of instrument used, the United States Geological Survey sends out during each field season an average of 120 plane-table parties equipped with telescopic alidades, the great majority being of the type shown in Fig. 1, Plate IV. The work of these parties is in all parts of the United States, Alaska, and the insular possessions; in other words, it comprises all forms of topography and climatic conditions. The United States Coast and Geodetic Survey likewise has annually from 25 to 30 plane-table parties in the field, covering the same range of territory and using an alidade of practically the same construction as that shown in Fig. 2, Plate IV. These are significant facts. Here are two scientific bureaus, independently of each other, pursuing work of a similar character. Through the experience gained during past years, not by a few, but by hundreds of topographers belonging to both Surveys, each bureau has evolved a type of alidade which seems to give the best satisfaction for its particular

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Mr. Matthes. work, and a comparison proves that the instruments are essentially alike, the principal difference being that the Coast Survey alidade is provided with a filar micrometer eye-piece, while that of the Geological Survey has either a plain or a prismatic eye-piece. These types of alidade have not been selected arbitrarily. Suggestions for improvements have been innumerable, and many of them have been given a practical trial. The present form of Government alidade has stood the test of years, and it may safely be predicted that nothing short of a radical change is likely to modify it.

A comparison of this alidade with Professor Webb's (Plate I, Fig. 1), becomes of interest. The most prominent difference, probably, is in the vertical arcs. It seems unfortunate that Professor Webb has selected, for the purpose of illustrating his attachment, an alidade provided with the old-fashioned arc, which requires the topographer to step aside at each reading. The arc adopted by the Government bureaus mentioned bears the graduations on the rim, that on the Coast Survey alidade being slightly beveled, and is adjusted so as to be easily read from the eye-piece end of the telescope; in short, it enables the topographer to read his angles without moving his position.

This is a feature of importance, for it not only saves him from stepping around the table, but frequently the table must be mounted on narrow projecting ledges where only one side of the instrument can be approached, and in such cases it is always possible to read the angle, even in difficult positions, as long as the eye can reach the eye-piece. A further advantage accrues from the fact that this form of arc permits a shortening of the Y's and a corresponding lengthening of the standard, a feature of prime importance in the rapid and safe handling of the instrument. A glance at the alidade on Plate I, Fig. 1, will convince one that not only is the standard awkwardly short, but the vertical arc and its bubble attachment are precisely where they are most likely to be injured, being alongside the part of the instrument which has to be handled most.

Professor Webb's adjustable vernier principle could just as readily be applied to the form of alidade shown in Fig. 1, Plate IV, and would there be in a much safer position.

The next and most important point of difference between the two forms of alidade lies in the method of reading the angle. The Government alidade has an arc, the graduations of which are entirely arbitrary as regards point of reference. Frequently, no zero point appears on the limb. Angles are read by subtracting the readings obtained with telescope inclined and telescope level, and, as will readily be seen, the topographer has no temptation to read an angle and neglect to correct it for the inclination of the table. Numerous devices have been experimented with to obviate the necessity of reading

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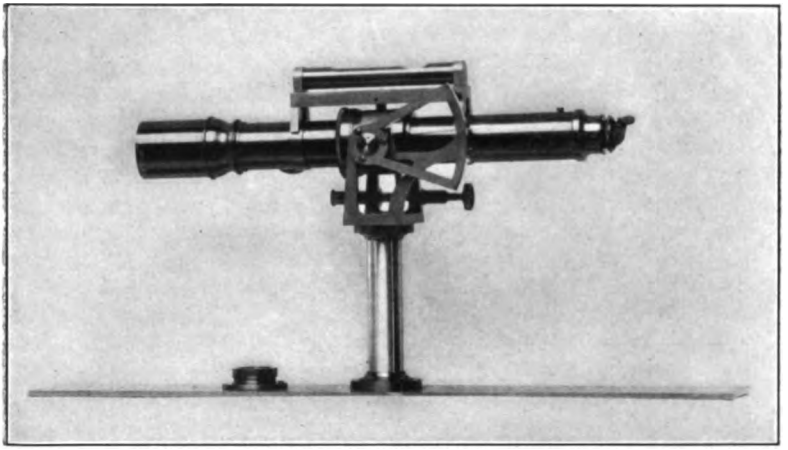

FIG. 1.—U. S. GEOLOGICAL SURVEY ALIDADE.

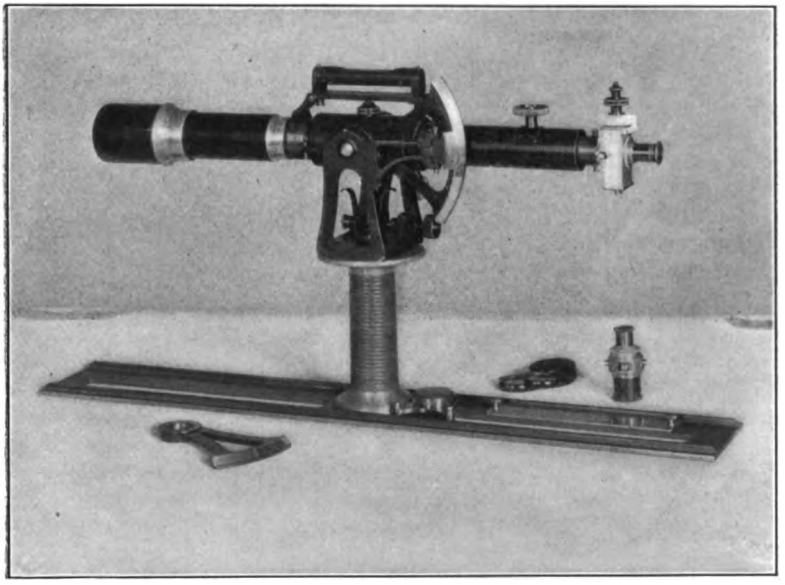

FIG. 2.—U. S. COAST AND GEODETIC SURVEY ALIDADE.

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the limb twice, and leveling the telescope after each pointing. The author's device is one of a great many, and is by no means new. Kern and Company, of Switzerland, who rank among the foremost instrument makers of the continent, have for years manufactured an alidade with an adjustable vernier almost exactly like that of the author. The principal advantages gained by this feature are: (1) The arc need be read but once; and (2), no angles need be subtracted. Thus time is saved in two ways. The objections are: (1) As stated before, an additional spirit level is required; and (2) the accuracy of the entire operation depends upon the assumption that the vernier reads zero when its level is parallel with the striding level and the latter with the line of collimation. Mr. Matthes.

In the writer's opinion, such a combination of conditions will require frequent testing of the adjustments, especially on hard trips. Besides, the adjustments involved number four (against two in the Government alidade), and one of them, *i. e.*, making two spirit levels parallel with each other, is a feat which even the author would probably hesitate to undertake when pressed for time, on top of a peak, assuming even a moderate wind to be blowing. The chances are that the average topographer would trust to the adjustment being correct, and proceed with his work, to the detriment of its accuracy. After all, in a practical consideration of the time actually saved by means of this device, it is found to be disappointingly small.

As regards doing away with the subtracting of angles, no time is actually saved, for the recorder has ample time on his hands in which to dispose of such computations, and check them besides. Doubtless, it would greatly lighten his work, and, in this sense, would be valuable, but no time would be saved by it as regards carrying on the survey. In this method, as in the other, a bubble must be brought to the center of its tube; and there remains, therefore, as actual net saving, the time required to read the vernier once. The question naturally arises: Is this saving of time commensurate with the addition of an extra spirit level? Evidently not; unless, perhaps, the alidade is to be used for detailed stadia surveys in centers of civilization, as, for instance, on plans for landscape architecture, where the means of transportation may warrant the use of elaborate instruments. Here, no doubt, the author's alidade will be found a most serviceable instrument.

A brief description of some of the best forms of alidades in use will be of particular interest in this connection. The Government alidade has only two adjustments, both of which can be instantly tested whenever the topographer has reason to suspect anything wrong; and the adjustments themselves are so simple that they can be disposed of in a very short time, varying, of course, with the skill of the topographer and the velocity of the wind. The first is the adjustment of the striding level, which is attained in the usual man-

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Mr. Matthes. ner, by reversing it, end for end. The second is the collimation adjustment, the horizontal wire only being corrected. For this purpose, the telescope has been fitted in a sleeve provided with stops which allow it to be turned 180° about its optical axis. The latest forms have a reticule held by four capstan-headed screws which can be moved in any direction. An error in the position of the vertical wire, however, will have no effect on the work, provided it is constant during any one set-up. The same is true of any lack of parallelism between the edge of the ruler and the line of collimation. The horizontal axis of the telescope is adjusted by the instrument maker as accurately as possible, and no provision is made for further adjustment. The limb is graduated to 30′, the vernier reading to single minutes. Most alidades are provided with a declinatoire fastened to the ruler, as shown in Fig. 2, Plate IV. In some classes of work a prismatic eye-piece is indispensable. When provided, as shown in the illustration, it should be made removable. To level the table a circular bubble attached to the ruler has been found to be the simplest and best device. The United States Coast and Geodetic Survey alidade is provided with a filar micrometer eye-piece, which enables the topographer to determine the distance to a triangulation signal by measuring the fraction of his cross-wire interval subtended by a known distance on the signal, a valuable aid in a country where triangulation signals can be built of wood. It is particularly useful in taking long sights, exceeding the limit of stadia observations, where no great accuracy is required, as, for instance, in plotting wood or marsh lines. A two-target rod is used in such cases.

Simple as it is, the Government alidade is entirely adequate for the work, and the very highest grade of topographic surveying is daily being done with it. On the whole, it may be said that it is much easier to design surveying instruments with a complication of parts, than it is to design those of simple and yet adequate construction.

It is gratifying and worthy of note that in this respect American instrument makers are ahead of European makers. Many types of surveying instruments manufactured abroad at the present day, referring to those of the best makers, are provided with parts, which, to the practical engineer in this country, are mere elaborations, and are frequently more of a hindrance than a help. Generally, many such parts, though useful, can be dispensed with altogether, by adopting different though equally good methods of prosecuting the work; or they could be replaced by simpler devices, which would reduce the weight of the instrument, increase its portability and necessitate fewer adjustments.

The error most frequently made is that of endeavoring to produce instruments that shall be capable of a multitude of operations, perhaps going to the extent of solving formulas, an endeavor which in

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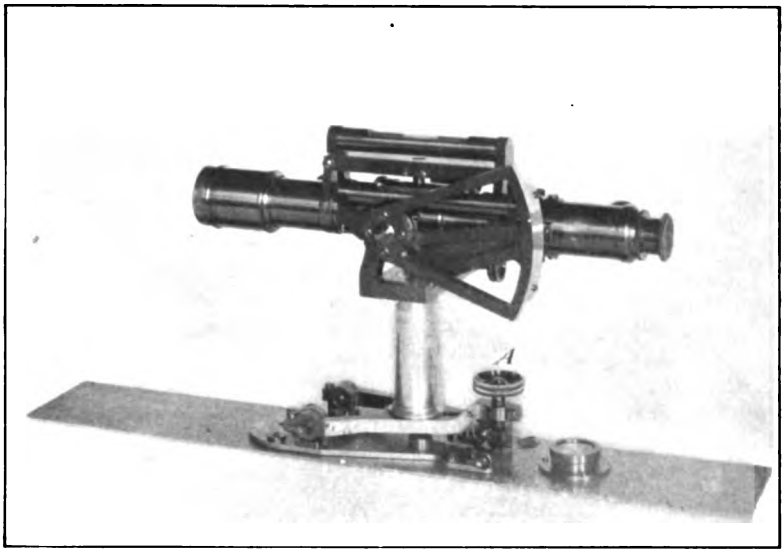

FIG. 1.—THE BUMSTEAD ALIDADE.

FIG. 2.—THE TACHYGRAPHOMETER.

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itself is commendable enough, having as a primary object the saving of time, but which in its practical application is likely to result in the instruments produced being so bulky, so unnecessarily complicated, and consequently so hard to keep in adjustment as to defeat the very object for which they were designed. As a conspicuous example of misdirected efforts of this kind, the Tachygraphometer, shown in Fig. 2, Plate V, and which will presently be described, is noteworthy. Mr. Matthes.

Among the great variety of existing forms of alidades, two are found which merit attention, being particularly designed to "increase the accuracy or rapidity of surveying operations." The Kern alidade, mentioned previously, is provided with a complete vertical circle graduated on its rim, with an adjustable vernier and spirit level, and with an attached reading glass. The telescope is mounted on one side of the standard, and can be completely revolved about its horizontal axis, its weight being counterbalanced on the other side of the standard by the vertical arc and its accessories. In addition, the standard can be fastened at several points along the ruler, a great advantage in working with a large table. The Kern alidade is a beautifully finished instrument, and its scarcity in this country must be accounted for by its elaborate character, which renders it wholly unfit for hard service.

Another form of alidade which unites many practical advantages is that invented by Mr. A. H. Bumstead, of the United States Geological Survey, shown in Fig. 1, Plate V, and manufactured for him by W. and L. E. Gurley and Company. As in the Kern alidade, its standard is detachable, and can be mounted rigidly and with ease, by means of an ingenious device on rulers of different lengths, carried by the topographer in a special case. The vertical arc is similar in form to that of the Government alidade, but is graduated 30° each way from the zero point, which is in the center of the limb. A spirit level, not easily distinguished in the illustration, being partially hidden by the vertical arc, is attached, by means of a stout projecting arm, to the standard, making it possible to level the entire instrument frame, exclusive of the ruler, by turning the screw *A*.

The adjustments are made as follows: The striding level is first made parallel with the line of collimation, as in other alidades; the vernier is then set at zero, and the entire instrument is leveled by means of the striding level and the screw *A*. This accomplished, the spirit level mounted on the standard is made parallel with the striding level, after which the striding level can be removed, and the alidade is ready for work.

The operation of reading a vertical angle is as follows: The telescope is pointed at the object to be located, and the bubble is brought to the center of its tube. The telescope is then clamped, and the horizontal wire is brought to the desired height on the object or rod, and the true vertical angle can then be read at once.

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Mr. Matthes. This alidade possesses another valuable feature, With the aid of a horizontal screw to the right of *A*, but not visible in the illustration, the entire instrument frame can be rotated in azimuth on the ruler. The object of this attachment is to enable the topographer, when first orienting the table, to bring the vertical wire exactly on the triangulation signal sighted, after he has brought it approximately there by setting and clamping the table. Mr. Bumstead's alidade has also been made with a filar micrometer eye-piece, intended to be used for the same purpose as in the case of the Coast Survey alidade. The former instrument has been used during the last two years by its inventor and others on the survey of the Adirondack Reserve, and has given excellent satisfaction. In the writer's opinion, the field for the above described forms of alidade, including that of the author, will necessarily remain limited, owing to the fact that their constructions are not adapted to rough means of transportation.

The Tachygraphometer, shown in Fig. 2, Plate V, is manufactured by L. Tesdorpf, of Stuttgart, Germany. It is designed to take the place of the alidade on the plane table, and purports to reduce and plat stadia observations mechanically. Being mounted on three ivory wheels, its position on the table can be shifted at will, small brakes being applied when the instrument is properly pointed. The telescope once set and the rod reading taken, the operator, for he can hardly be called a surveyor, sets the index of a scale opposite the proper rod reading, and without further bothering his head about vertical angles, proceeds to shift a series of parts, when lo! the elevation of the point sought is indicated on the vertical scale, and the location of the point itself is correctly brought about by merely pressing a button. Had only a nickel-in-the-slot attachment been provided, could anything more perfect be imagined? But let not the reader enthuse too soon. The operator has completed his set-up and is about to move to another point. He takes the Tachygraphometer from the table, but finds he cannot carry it like an alidade, for not only is it too heavy, but it affords no convenient hold anywhere. He must put it back into its box. More trouble! The instrument will not go in as it is; some of its parts must first be removed. These fit in the box in places specially provided, and not until they have been taken care of can the instrument itself go in. The box is ready at last, and a right heavy load it is. The topographer picks up his table, the assistant with the Tachygraphometer comes toiling after, and by the time they reach the next station, they will probably find their patient rodman slumbering peacefully. The stadia rod used in conjunction with the Tachygraphometer is held at right angles to the line of sight, the rodman being provided for this purpose with a simple wooden attachment serving as pointers. In keeping with the complicated character of the instrument, the graduations of the rod are of a complexity that defies description.

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The writer has had occasion to test the Tachygraphometer. It did splendidly, but fortunately he did not take it further from the house than the adjoining tennis court. Only one instance of this instrument being used on actual surveys on this side of the Atlantic is known to the writer. This was on a railroad survey in Mexico, during the course of which it soon developed that the Tachygraphometer was least of a nuisance when left at the office. If other surveyors have had a profitable experience with this instrument the writer would be glad to be informed. The Tachygraphometer is a beautiful mathematical machine, but, as a surveying instrument intended to lighten and dispatch work, it is a failure. Mr. Matthes.

C. A. SUNDSTROM, M. Am. Soc. C. E. (by letter).—Mr. Webb's devices no doubt increase the accuracy of the work, but some of them introduce great refinement, where only approximate results can be obtained, and, by increasing the number of appurtenances to the instruments, make them unnecessarily bulky and heavy. Mr. Sundstrom.

As the plane-table alidade, the first instrument mentioned by Mr. Webb, must be used in conjunction with the plane table, a short review will be made of the uses for which the plane table is adapted.

The main object of all surveying instruments is to determine the geographical location and shape of tracts of land or objects on the surface of the earth. The principal outlines of a tract of land cannot be determined accurately by means of a plane table. They must be located geodetically or trigonometrically. But the interior part, or the filling in, can be accomplished by the use of the plane table. The location of houses and other objects on the surface of the earth can be made very advantageously by the method of intersections, when the alidade without telescope can be used. For the location of contours, a more complicated alidade can be made useful. In the writer's opinion, leveling with the alidade is not practicable. The best results are obtained by using an engineer's level for getting elevations, and using the alidade only for determining geographical positions. The alidade will then be reduced to the smallest number of parts, and thus become light and convenient to handle. A light alidade will not be so apt to throw the plane table out of line as a larger and heavier instrument would.

The writer was employed by the Swedish Government on topographical work about 24 years ago. On this survey, all the instruments were extremely light and convenient to handle, with the exception probably of the theodolite used on the triangulation. The plane table was made in the form of a box, 14 x 12 x 1½ ins. The bottom and sides were made of hard wood, beech or birch, and the top of white pine or aspen. One longitudinal and one transverse rib added strength to the table. The thickness of the wood was ¼ in.

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Mr. Sundstrom. The top was made of soft wood, so that a needle could be used to determine the point about which the alidade was to revolve; and, in order to secure the exact position of the alidade along important sight lines, a needle was placed near the edge of the table, at each terminus of the line. The appurtenances to the plane table consisted of two alidades (one with and the other without a telescope), one spirit-level, one compass and one plumb-bob, all placed in a light box, which the rodman carried on his back by a strap. The alidade, with the telescope, weighed less than 3 lbs. The telescope was $\frac{3}{4}$ in. in diameter, and was fastened to the side of a hollow cylindrical post, attached to a ruler, 12 ins. long and $1\frac{1}{2}$ ins. wide, beveled on one edge, and channeled on the under side. The telescope was provided with a level, a vertical arc and stadia wires. The paper used for the plane table was 14 x 16 ins., and was stretched on the table in the following manner: The four corners were cut off, the cuts making right-angled triangles of about 1 in. base and altitude, each hypothenuse being perpendicular to the diagonal through the corner. The paper was then wetted, and, after glue had been applied to the sides of the plane table, the overhanging edges of the paper were turned down and glued to the sides of the table. When the table was nearly covered by parts of a map, an extra table, with the paper stretched on it, was carried in the field, and upon it the work was continued. When more than one extra table was used, a light box containing them was carried along.

The nearest approach to Swedish plane tables, which the writer has seen in this country, are those used by the United States Geological Survey, and are 15 ins. square.

The use of the stadia slide rule is not so convenient as the graphical chart. Such a chart, made of the same size as the plane table, can easily be placed on top of the table and held there by the covering used for the protection of the table, while the instrument is moved from one station to another. The chart has another advantage in enabling one man to attend to the plane table, without losing time. As soon as the stadia reading has been made, the plane-table man has ample time to make his reduction by the chart, and plot the point on the table, while the rodman moves to another station and the leveler to a new point. A self-reading level rod was used, both for leveling and stadia readings.

The tension frames for steel-tape measurements are not practical. No doubt, they work well in measuring a base line on the university campus, but they are useless in hilly country. Professor Jäderin's arrangement* is far better.

Before leaving this subject, the writer would like to call attention to some devices which he has found useful during many years' practical

*See Johnson's "Theory and Practice of Surveying."

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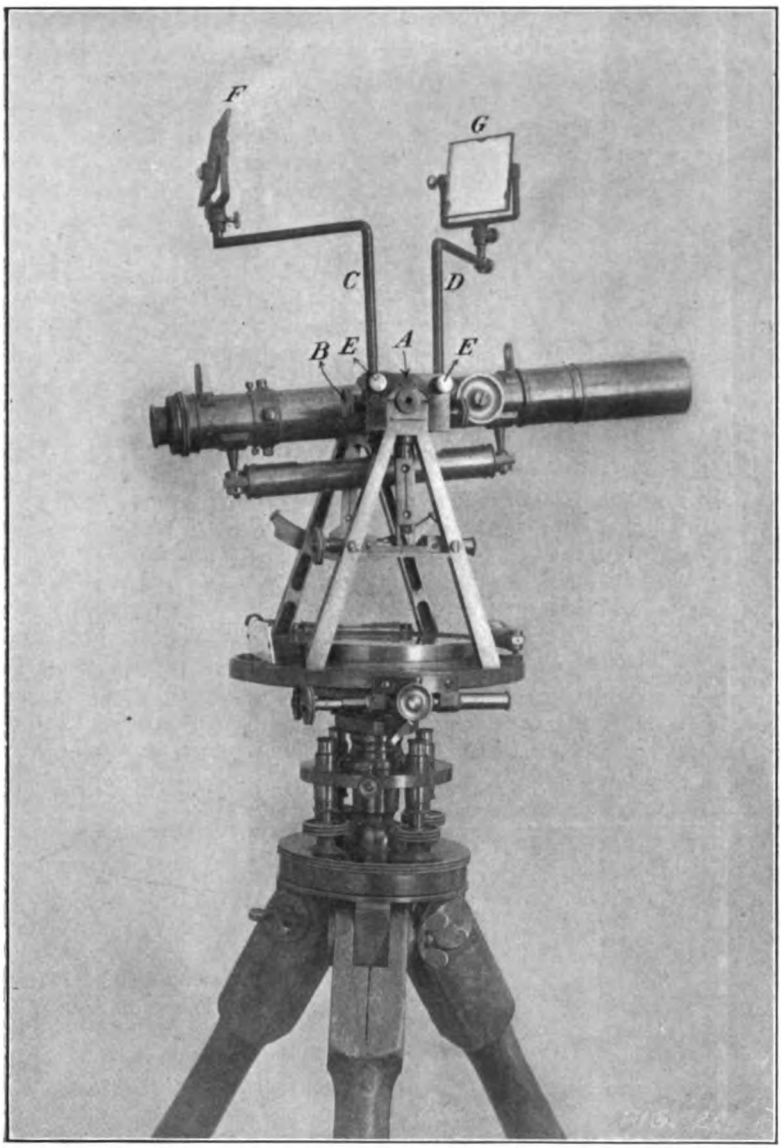

HELIOTROPE ATTACHMENT FOR A TRANSIT.

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experience in surveying. One of the most useful is the etched cross-hair, made by photographing a cross on a thin piece of plate glass, large enough to be placed in the metal ring which usually carries the cross-hairs. After the negative is obtained, it is exposed to the fumes of hydrofluoric acid, until an etching sufficiently deep has been produced. In order to make the cross opaque, blacking can be placed in the etching, which makes the cross more clearly visible when strong light enters the objective of the telescope. Mr. Sundstrom.

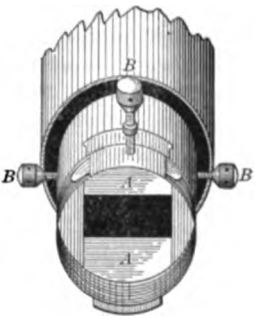

FIG. 2.

The glass-edged stadia are made from two pieces of plate glass, attached to a metal frame, as shown in Fig. 2. The pieces of plate glass, *A A*, are adjusted by the capstan screws, *B B*, which operate the metal frames into which the glasses are inserted. The glasses have their inner edges slightly beveled and highly polished, and, after having been properly adjusted, the beveled edges will serve as stadia wires. An arrangement is also made, by which the glass-edged stadia are out of focus when the cross-hairs are visible, and *vice versa*. A transit with etched cross-hairs and glass-edged stadia has been in use by the writer for two years, and, although tested frequently, there has been no necessity for adjustment in that time.

The heliotrope attachment to a transit is shown on Plate VI. *A* is a clamp attached to one of the standards of the transit, and is held in position by the screw, *B*. Two bent standards, *C* and *D*, are inserted in the clamp, *A*, and secured by the screws, *E E*. Each of the standards carries a mirror with two motions, and, by revolving the standards and moving the mirrors, a flash in the direction of the line of collimation of the transit can always be produced in every position of the sun.

JOHN F. HAYFORD, Assoc. M. Am. Soc. C. E. (by letter).—In the fifth portion of Professor Webb's paper it is set forth rather tersely that to satisfy the requirements of modern steel-tape measurements it is necessary to determine the tension on the tape with great accuracy; that the tension must be determined "with an accuracy which is beyond question"; that the ordinary spring balance is useless for this purpose, and that even if a high-grade spring balance is used there is an uncertainty as to the tension actually measured when the balance is used in the horizontal position, which, added "to the feeling of uncertainty of the measurement of a force by a spring" makes the spring balance unfit for use in connection with precise steel-tape measurements. An apparatus designed by Professor Webb for putting the tension on a steel tape and measuring it by the moment of a weight about a knife-edge is then described fully. Mr. Hayford.

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Mr. Hayford. The writer does not believe that the facts warrant such a summary dismissal of the spring balance from consideration. It has proved its usefulness and reliability on twelve primary bases in the Coast and Geodetic Survey, and has remained and still remains in use and in good favor after ten years of service. During all these years the spring balance and its mounting, as used by Professor R. S. Woodward on the Holton Base,* has remained practically unmodified. It will be difficult to find in the United States any other stretching device for tapes which has been subjected to a test which is comparable with that through which this one has passed so successfully.

The writer believes that the method of measuring the tension on a tape which is advocated by Professor Webb—and it has been advocated by others—has a possible field of its own. To adapt the device for actual field use, rather radical changes in the design here submitted would seem to be necessary, however.

In the following paragraphs an attempt will be made first to indicate briefly the superiority, under certain conditions, of the spring balance over the gravity balance for the measurement of tape tension; and secondly to indicate the possible field within which the gravity balance is to be preferred to the spring balance.

The spring balance used by the Coast and Geodetic Survey in its primary-base measurement carries a dial graduated to 25-gr. spaces. Under field conditions there is found to be no difficulty in holding the pointer to within 12 gr. (0.4 oz.) of a given reading. This spring balance is hung in gimbals at its center of gravity (after counterweighting). The gimbals are supported in position by a lever about 4 ft. long used in a nearly vertical position and having its lower end connected by a universal joint to a heavy foot-board laid flat upon the ground, and upon which the operator stands. The gimbals and balance can be adjusted quickly to any desired height within a total range of about 2 ft. by a heavy nut moving on a screw thread cut on the lever. With this apparatus an experienced operator requires very few seconds, after reaching a new station, to get his balance adjusted to the proper height and position and to apply the desired tension. Neither does he find any difficulty in maintaining the desired tension continuously within 12 gr., even at the end of two or three hours of continuous tape measurement during which he has walked and carried his apparatus several miles, and set it up at scores of different points.

All accurate field tape measurements, in so far as the writer is aware, are made with the tape supported at a few points only. Under these conditions the distance between the end graduations of the tape increases with an increase of applied tension, both because the tape stretches by an amount which is dependent upon its modulus of elasticity, and because the sag of each catenary formed by the tape is decreased.

* See Coast and Geodetic Survey Report for 1892, Appendix No. 8, pp. 414-415.

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Mr. Hayford.

The primary-base tapes now used in the Coast and Geodetic Survey are of steel, about 6 mm. ($\frac{1}{4}$ in.) wide and 0.5 mm. ($\frac{1}{75}$ in.) thick, and weigh about 22 gr. per meter of length. When in use they are supported at intervals of 25 m. The tension applied is 15 kgr. A variation of 12 gr. (0.4 oz.) in either direction from the tension of 15 kgr. makes a change of only 1 part in 3 000 000 in the apparent length of the tape. As previously stated, the tension is easily held within this limit on each tape length. The small outstanding accidental variations from the standard tension, due to variations within this limit, and to friction in the apparatus, necessarily have a decided tendency to be eliminated by compensation in the long run. All difficulty about the difference between the inclinations of the balance in the horizontal and the vertical positions is avoided, in the practice of the Coast and Geodetic Survey, by standardizing the balance in the horizontal position. One of several spring balances is carefully standardized at the beginning and end of the working season. During the working season the balance which is actually used on each night of base measurement is compared with this standard just before starting away from camp. The same man operates the spring balance throughout the season, during the standardization of the tape as well as during the base measurement. As what is really necessary to secure accurate results is that the tension applied to the tape during base measurement shall be exactly the same as that applied to it during standardization, it is believed that the practice outlined above insures that the constant error in any base due to error in applied tension is surely less than 1 part in 1 000 000.

In the primary-base measurements of the Coast and Geodetic Survey the device for applying the tension at the rear end of the tape is simply a light crowbar. The man with the bar comes up to his new position when the tape is carried forward, strikes the bar into the ground at the proper point, which he estimates with sufficient accuracy by eye, slides the loop of cord, around the bar to which the tape is fastened, upward or downward to the desired position, and by a backward movement of the upper end of the bar, using it as a lever, brings the graduation of the tape to exactly the correct alignment and longitudinal position. An officer with a small reading glass, kneeling at the rear-end stake, controls the final adjustment to exact coincidence between the tape graduation and the fine line on a copper strip on the stake which marks the previous position of the front-end graduation of the tape. He controls the man at the bar by word, and, at the same time, steadies the tape and eliminates slight longitudinal motions by pressing the tape lighly against the top of the stake with one finger. By this method there is found to be no difficulty in making a coincidence to within 0.1 mm. (or $\frac{1}{250}$ in.) in a few seconds.

No appreciable gain in accuracy in geodetic operations is gained by reducing the accidental errors of base measurement below the

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Mr. Hayford. limit indicated by a probable error of 1 part in 500 000. Any greater accuracy is lost in the first group of triangles surrounding the base. If steel tapes are used with mercurial thermometers at night, there is no difficulty in rapidly securing as great a degree of accuracy as this. On the other hand, even if all possible precautions are used, the probable error can with difficulty, and only at considerable extra expense, be reduced below 1 part in 3 000 000 when mercurial thermometers are used, and great and perhaps insuperable obstacles are encountered in trying to insure that a constant error as great as 1 part in 1 000 000 is not left in the final result, due to the thermometers indicating an erroneous temperature for the tape. The writer believes that within this range of accuracy, of which the lower limit is fixed by the possible accuracy of triangulation and the upper limit is fixed by the defective indications of mercurial thermometers, the spring balance is to be preferred to a gravity balance as a tape stretcher, because it can be much more conveniently and quickly handled.

Professor Webb's gravity balance would be found to be very slow and inconvenient to handle in the field, because it has insufficient range of adjustment and very slow means of adjustment. The base tape must hang clear of the ground at all points. This makes it necessary to have a range of at least 2 ft. in the height of end stakes, to avoid much grading. There must be a corresponding range in height of adjustment of the stretching device, and the device must be capable of a quick adjustment over this whole range. Professor Webb's apparatus is decidedly faulty in this respect. A successful tension apparatus must have within itself a considerable range of longitudinal adjustment, otherwise much time will be lost in placing the supporting frame by trial in various positions, especially in stony or otherwise troublesome ground. Both the front and rear end tension frames in Professor Webb's apparatus are decidedly defective in these respects, in comparison with the Coast and Geodetic Survey apparatus. Moreover, in the use of Professor Webb's apparatus it is evident that the inclination and the alignment of the tension frame at the front end of the tape must be made nearly correct, as a comparatively small error in either of these respects will either throw the apparatus out of use or make the tension erroneous. The necessity of setting the front tension frame to conform with these requirements will help to make it a slow and annoying apparatus for field use. In the Coast and Geodetic Survey stretching apparatus these two adjustments are made automatically by the gimbals.

During the working season of six months in 1901 nine primary bases were measured by the Coast and Geodetic Survey by one party, under Mr. A. L. Baldwin.* These nine bases are scattered along the

* A complete account of these base measurements will appear in a few months in Appendix No. 3, of the Coast and Geodetic Survey Report for 1901.

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Mr. Hayford.

ninety-eighth meridian from northern Nebraska to southern Texas. Their aggregate length was 69 km. (43 miles). The probable errors of these bases vary from 1 part in 690 000 to 1 part in 1 640 000. About four-fifths of these measurements were made with steel tapes, mercurial thermometers and the stretching apparatus referred to previously. About two-fifths of all these measurements were made with tapes only 50 m. long. The average speed of measurement with these 50-m. tapes for the whole season was 1.8 km. (1.1 miles) per hour, or thirty-six tape lengths per hour. On eight different occasions during the season measurements of from 1 to 3 km. were made at a speed exceeding 2 km. per hour. This corresponds to less than 90 seconds per tape length, including the walking and miscellaneous delays. The time spent, after reaching a new tape position, in stretching the tape, making the coincidence at the rear end, marking the position of the front end, and reading the thermometers, was, therefore, during regular work, from 30 to 40 seconds, on an average, and, very frequently, must have been less than 20 seconds. If Professor Webb will try to manipulate his apparatus, even with an experienced party, at a speed even one-half as great as this, over ordinary ground, he will understand why the Woodward tape stretchers are still in favor in the Coast and Geodetic Survey.

If any method is put into use by which the temperature of the tape can be obtained accurately, and the limitation on the accuracy imposed by the use of mercurial thermometers is thus removed, it will become possible to measure a base with a steel tape with a probable error of 1 part in 5 000 000. When this is attempted it will be necessary to measure the tension with a considerably higher degree of accuracy than is attainable with the spring balances and stretching device now in use in the Coast and Geodetic Survey. The gravity balance may be the tension instrument of great accuracy which is needed for this purpose. But it is not at all certain, if equal care were taken in designing two sets of stretching apparatus for this special purpose, in one of which a gravity balance should be used and in the other a specially designed spring balance, that the spring balance would not be found to give as great accuracy as the gravity balance and still preserve its present superiority as to rapidity and convenience of handling. It should be kept in mind that the spring balance now used by the Coast and Geodetic Survey is of the ordinary type which can be bought in the market for $3.50, and it will not be difficult to improve upon its present construction. The writer would be pleased to see someone perfect the gravity balance for use as a tape stretcher on field measurements of the highest grade of accuracy. Professor Webb's design would be unsatisfactory for such use unless radically modified along the lines indicated by the preceding criticism, so as to make it capable of rapid handling.

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Mr. Hayford. In the preceding paragraph the possibility of a method of determining the tape temperature with the accuracy necessary to obtain a probable error of 1 part in 5 000 000 being put into use is mentioned. The thermophone apparatus, designed at the Massachusetts Institute of Technology,* furnishes, so far as the writer is aware, the only promise of immediate success in this respect. This apparatus measures directly the variation in the electric resistance of the tape, which is a function of the temperature of the tape.

Mr. Lockwood. WILLARD D. LOCKWOOD, Assoc. M. Am. Soc. C. E. (by letter).—This paper has brought forward some points that have been often discussed in the hearing of the writer during the past ten years, and which are probably familiar to most engineers who have had any very extended experience in the field.

The writer agrees most cordially with Mr. Webb as to the use of the stadia in connection with the plane table, though the experience of the writer, with the plane table, is where it has been used as a means of cutting in controlling points, with the intermediate topography sketched in; but Mr. Webb evidently has reference to more detailed topography.

As to the accuracy of stadia work when the plate bubbles alone are to be depended upon, any intermediate point on such work, for a topographical map, is assuredly secured by the plate bubbles much within the limit of control of the shot in question. Take a shot of 1 000 ft.; of course, on a stadia survey there must of necessity be many such, as the great value of stadia work is in covering large areas from few set-ups. Now, in 1 000 ft., where the intermediate shots are being taken with great rapidity, the rod can be moved forward or back 1 ft., and the observer would seldom make a change in the reading, thus making a variation of 2 ft. in distance; now, suppose the ground to be very irregular, or a side-hill, then this 2 ft. could easily change the elevation 1 ft.; hence it is quite evident that a minute of arc in elevation is a small factor, as in 1 000 ft. it would amount to about 0.3 ft.; and the writer has observed that the horizontal corrections on the intermediate shots are seldom made when the angle is less than 4° (as it is evident that it would be useless refinement, from the fact that the contour to the next point is to be sketched in, and the uncorrected shot in question will be well within the limits of the work). This is all with the understanding that the survey is controlled by a transit, level and steel-tape base line, which the writer believes should always be the case where close map work is to be expected.

It would seem to the writer that the refinement suggested by Mr. Webb is rather unnecessary; past experience has shown that splendid results may be achieved by the regulation instruments, when the work

* A description of this apparatus is published in the *Technology Quarterly*, June, 1901, pages 82–88.

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is done with care by good men. In the report of D. J. Howell, M. Am. Soc. C. E., to the United States Board of Engineers on Deep Waterways, he states that with properly graduated rods and ordinary refined methods, the following results were obtained: Mr. Lockwood.

Forty-four circuits, selected at random from several months' work, as run between April and July, 1898, show

Average length of circuits	7 384 ft.
Average number of stations occupied	12
Average running error in azimuth	1' 32"
Average error in elevation	0.35 ft.
Average error in circuit	1 in 1 832
Maximum error in circuit	1 in 760
Minimum error in circuit	1 in 4 244

Of course, these results speak for themselves.

As to the vertical arc on a transit, the writer has heard more hard names applied to a transit, and to the company who sent it, when it appeared on the work with an arc instead of a full circle, than to all the other cheap instruments and attachments ever sent out. And the adjustable vernier secured nearly as much abuse as the arc, and to the mind of the writer, justly so, as many mistakes have been made through the use of this form of attachment, it being doubtful what class of work a man would do with it who could not put an instrument away in the box, as Mr. Webb suggests, without bending the arc; such a man being just about as apt to knock the capstan heads off the diaphragm screws.

As a rule, when an engineer has a survey of magnitude to make, he finds there is but little money with which to make it, as the money comes later, when the project is decided by the survey. What he desires to get are plain men and plain instruments, both without attachments; not necessarily the much-vaunted "practical men," but the men of that class who have been so pertinently described as "men who can do things." The points of control for the map are first secured by using the combined strength and intelligence of his whole party; then the filling in can be done by any of the various methods, without a possibility of multiplying errors. These methods are used, it must be understood, when material for a first-class map is wanted, not for an elaborate, padded report.

As to the carrying of a transit through the bush, the writer has found that his associates always agreed that there was but one safe way, and that was with the telescope thrown up in line with the tripod, as when allowed to project it will indeed be a "sore finger," as Mr. Webb says of the arc. Of course, this position of the telescope, with an arc, is out of the question, and is one of the many reasons against the use of anything but a full circle.

In connection with the subject of "Aids to Field Operations," the

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Mr. Lockwood. writer would like to mention the hand-level, as one cannot be thoroughly familiar with the great value of this little instrument without having had extensive operations in a heavily wooded country. In the course of recent surveys made by the writer, under the Isthmian Canal Commission, in Nicaragua, the usefulness of the hand-level was shown up in all its details. In making a location for a part of the canal line, it became necessary to develop a large area of hilly country, the most densely wooded in the world, where nothing more than tunnels could be cut through the rank vegetation. A base line was run with transit, steel tape and wye-level, and then the topographers with their little hand-levels and cross-section books attacked the maze of tunnels and cross-tunnels, rarely seeing the sun from the time they left camp until their return. The writer believes that the method of cutting the map into small sheets and actually plotting the topography as it is taken in the field, is much superior to, and fully as economical as, the use of note-books. In Nicaragua it was out of the question, owing to the constant rains, as the note-books by night were often as wet as though they had been dropped into a river, and, in fact, they occasionally were, along with the topographers.

The results of the hand-level topography were most gratifying, as a projection made on the map, from this survey, was staked out with precision, and checked a line through the work several miles long.

It has given the writer pleasure to see the details so well worked out in Mr. Webb's tension frame for steel-tape measurements, as the old devices, of which the writer has made use in the past, compare about as favorably as an old stage coach to a modern automobile. But, in making use of Mr. Webb's device, it would seem that more ease in manipulation could be secured by extending the graduated lever-arm forward, with a tip turned up at the end, to correspond in height to the tape clutch above the arm, thus readily showing when the tape is in perfect alignment, and also pulling absolutely at right angles to the short lever-arm.

Mr. Sabin. L. C. SABIN, Assoc. M. Am. Soc. C. E. (by letter).—The increasing use of steel tapes or wires for accurate measurements lends a special interest to that part of this paper relating to the method of applying the tension to base-measuring tapes.

The author's adaptation of the principle used on the Missouri River Survey in 1885, by the late O. B. Wheeler, M. Am. Soc. C. E., for this purpose, has many convenient and ingenious features. By improving the method of attaching the tape and weight to the bell-crank, and by providing means for making the small adjustments required to bring the tape in position over the stake, the author has made an improvement over the old wooden triangle with a knife-edge in the corner; but it would seem that he had stopped short of providing a practical instrument for actual work under any but exceptional conditions. The

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adjustment for the height of the tape above the ground seems to be very limited, the actual limit depending upon the length of the legs of the tension frame, and the softness of the ground. If the earth is so soft as to allow these legs to be pressed into it any great distance it is certainly not ideal for accurate measurements, and it is seldom that one finds the ground so level that he can avoid a variation of 2 or 3 ft. in the height of the end marks of the tape without frequent changes of grade. This, however, is a detail that might be remedied; the real question is: Can this style of tension appliance be made sufficiently portable to justify its replacing the spring balance? Mr. Sabin.

The possibility of the introduction of errors through the use of the spring balance cannot be denied, yet with proper precaution it is capable of giving quite accurate and reliable results. It is a comparatively easy matter to test the constancy of the balance with a known weight, though, as the author states, to determine the actual tension on the tape an allowance must be made for the difference between the readings of the balance when vertical and when horizontal.

The steel tape is capable of giving results of considerable accuracy, without great refinements in the preparation of the line and the method of use. It finds its greatest usefulness in this field, and it is not wise to multiply contrivances, unless a distinct gain in accuracy is made. With a spring balance and a tension frame of wood, that may be made in a few minutes, the writer has measured short bases with such accuracy that the error of the first quadrilateral off the base was greater than the portion of the error in length of base which could be charged to the manipulation of measurement. While no single step in the method is thought to be new or original, as it is taken in the main from Professor Jäderin, it may be described, as a whole, to show that sufficient accuracy for most purposes may be attained with very simple appliances.

The tape is supported by wire nails driven horizontally in stakes placed 25 ft. apart along the line. The stakes to hold the end marks are 2 x 4 ins., well driven. The mark at either end of the tape is referred to one edge of the head of a small brad, driven in the top of the 2 x 4-in. stake so that the head is nearly flush with the head of the stake. The alignment and grade are determined in the ordinary way. The tension frame or tape stretcher consists of a wooden handle, 5 ft. long, hinged to a short piece of 2-in. plank forming a foot-plate. The operator places the foot-plate a short distance beyond the stake, at the forward end of the tape, and stands on it, facing the tape. The hook of the spring balance is attached to the tape, while the ring at the other end of the spring balance is fastened to the handle of the stretcher by a wire loop which may be moved up or down on the handle, thus permitting the elevation of the tape to be adjusted quickly to any ordinary height of stake. The operator has a staff

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Mr. Sabin. which he thrusts in the ground some distance back of the forward stake, and grasps both staff and handle, thus forming an A. This enables the operator to give a steady pull on the tape. The rear end of the tape is held by a similar stretcher without the spring balance.

Six men are required for accurate work, as follows: Two observers, one recorder, two men to run the stretchers, and one to read the thermometer at the 50-ft. mark and assist in carrying the tape forward. The operator at the rear end is directed by the observer at that end, while the recorder stands directly over the spring balance, and directs the forward operator. When the 100-ft. mark of the tape is even with the edge of the rear brad, say the forward edge, the rear observer calls "read" and if at that instant the spring balance gives the correct indication, the recorder repeats the word and the forward observer reads, on an ordinary celluloid-edged scale, the distance from the zero end of the tape to the forward edge of the brad in the forward stake. This reading (usually taken in five-hundredths of a foot) is repeated as many times as desired, not less than three, and recorded with the thermometer reading. The tape and stretchers are then carried forward 100 ft. and the operation repeated.

One base of 22 tape lengths was measured three times in June with one tape and three times in November with a different tape. The actual time required for each measurement and the indicated lengths of the base are shown in Table No. 1.

TABLE No. 1.

Measurement number.	Time of measurement, in minutes.	Temperature, in degrees, Fahrenheit.	Reduced length.
1	76	69	2181.827
2	60	68	2181.926
3	45	66	2181.892
4	48	46	2181.862
5	36	47	2181.848
6	40	48	2181.847
			Mean = 2181.867

$$r = 0.024; \quad r_o = 0.01 = 1 \text{ in } 218\,000.$$

The work should be carried on rapidly, for the accuracy of the measurement depends upon the fidelity with which each stake in succession holds the end mark while the tape is carried forward. There is no place or need for "leisurely" determinations.

If the spring balance is considered too unreliable, then it would seem that a small wheel furnished with ball bearings would be more easily manipulated than the bell-crank. A wire connecting the tape with the weight would pass over a groove in the circumference of the

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wheel, and the vertical adjustment of the wheel would be provided for by an adjustable A-frame having several pairs of grooves to receive the axle. Mr. Sabin.

When tapes or wires are used in the measurement of primary bases, where extreme accuracy is desired, the use of the spring balance cannot be recommended. In the compound-wire base apparatus of the United States Lake Survey the spring balance was tried and discarded for the ball-bearing wheel. As used thus far, the wires of the apparatus are so long (1 km.) that much time can be spared for preparing stable end marks, and for setting up the apparatus, one tape length being all that is ever measured in one day. The tension wheels of this apparatus seem to leave little to be desired, in point of accuracy, but as the wheel supports are designed for use with either six or twelve wires side by side, they are much too unwieldy for a portable apparatus. All the accuracy pertaining to the use of the wheel, however, could be realized for one tape by the simple appliance just mentioned, and the advantage of having the tension automatically maintained without the use of thumb-nuts or levers is at once manifest.

Oscar Erlandsen, M. Am. Soc. C. E.—The subject of this paper is of interest to almost every member of the profession, because, whatever may be his special field of activity, he has at one time or other had experience in precise surveying; moreover, the field covered by the paper is one in which great improvement has been made during the past few years. Perhaps not much more can be expected in the way of increased precision, but there is room for further improvement in the reduction of the time consumed. Mr. Erlandsen.

During the past few years the Department of Bridges of the City of New York, in connection with the surveys for the various bridges over the East River, has developed devices intended to decrease the time required without decreasing the degree of precision. As will be shown, these devices compare very favorably, in precision of results, with those described in the paper; while in simplicity and flexibility or adaptability to conditions, varying all the way from those found in the crowded down-town business districts to those found in the practically unimproved and uninhabited portions of Long Island City, they compare even more favorably.

The tape used is a steel ribbon, $\frac{1}{8}$ in. wide, $\frac{1}{40}$ in. thick and 200 ft. long. It is provided with a spring balance at one end and also with a turnbuckle at each end for the regulation of the tension. The temperature for each measurement is obtained by means of a mercurial thermometer. The thermometer and spring balance were both tested at Washington and found to be correct. The tape is supported at intervals of 25 ft., under a tension of 12½ lbs., but the points of support are not necessarily level. All that is necessary is that they shall all be on the same grade for any one tape length, but it is not necessary

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Mr. Erlandsen. that this grade shall be the same for consecutive tape lengths. Moreover, the points of support are only temporary, and are set up anew for each measurement. Stakes, therefore, are dispensed with. To meet the requirements of supports that can be readily set up and adjusted to any grade, the apparatus shown on Plate VII was devised. The intermediate supports are wooden rods, 1 in. thick by 2 ins. wide, shod with steel points. They are usually and conveniently aligned with a transit. Each rod is held by means of a special extension-leg tripod, the head of which is provided with two pairs of opposing thumb-screws. Along the center line of each rod is a line of holes $\frac{1}{2}$ in. apart. Pegs are placed in these holes in the rods on the grade of the end supports, the position of each being determined by eye, by sighting over the end supports. The holes in the rods being $\frac{1}{2}$ in. apart, each point of support is within $\frac{1}{4}$ in. of the true grade.

To each end of the tape is attached a piece of light-weight plumber's brass chain, which passes over the end support to a pin driven into the ground, as shown on Plate VII. The end supports are adjustable wooden crotches. Along the center line of each of the two pieces forming the crotch is a line of holes which, with a small bolt, provide for the adjustment of the crotch at any desired height. To make a rigid frame of the crotch, the lower ends are hinged to a connecting bar. The end of each measurement is marked on an adjustable table specially designed for the purpose, as shown on Plate VII. Its top is a steel plate, 6 ins. square, on the upper side of which is pasted a sheet of paper, on which the end of the measurement is marked with a sharp pencil. On the lower side of the plate is a hollow cylindrical lug, which fits over the top of an iron bar. A thumb-screw in the lug permits of clamping the table on the bar at such height that it just touches the tape. Where possible, the bar is driven into the ground. In other cases the bar is set in a heavy cast-iron base and clamped. The elevations of the steel tables are obtained by means of the ordinary engineer's level and rod.

The following examples will illustrate the degree of precision attained. The base of the triangulation for the Blackwell's Island Bridge was measured four times in December, 1900. The results, reduced for grade and temperature, were—

1671.030 ft.
1671.028 "
1671.028 "
1671.060 "

The last was discarded because high winds prevailed while it was being made. The variation of the other three from their mean is less than 1 in 2 000 000.

The distance between two monuments in Long Island City on the center line of the same bridge was measured in January, 1901, and

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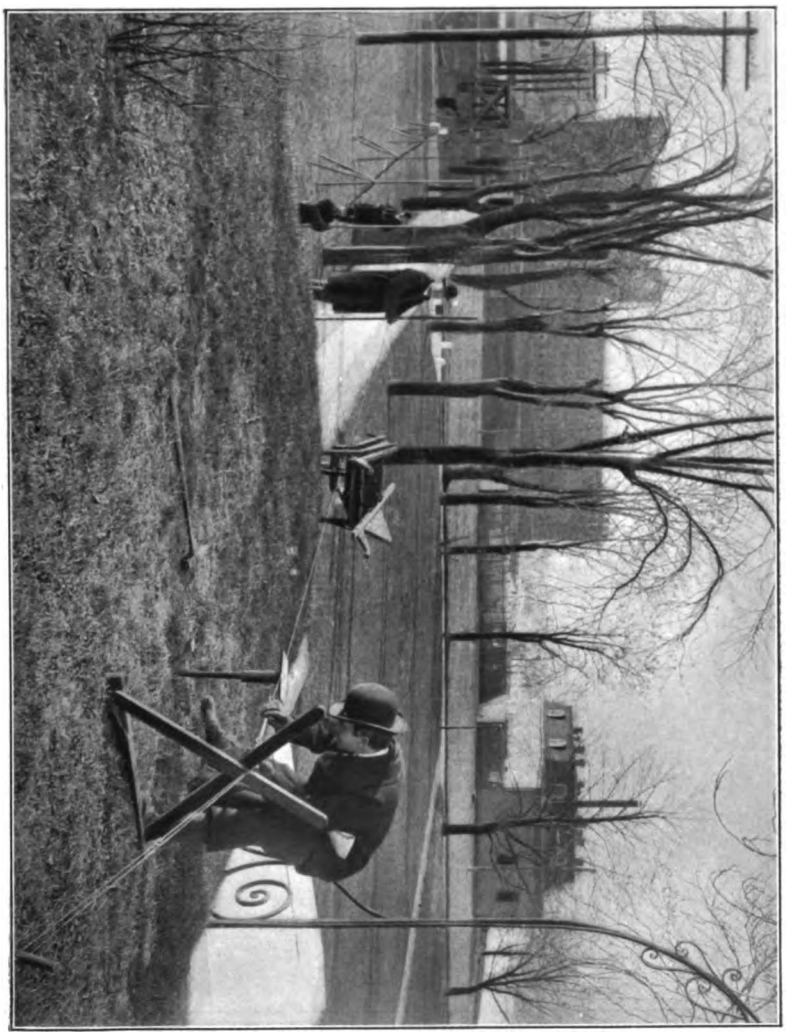
PLATE VII.
TRANS. AM. SOC. CIV. ENGRS
VOL. XLVIII, No. 919.
ERLANDSEN ON SURVEYING INSTRUMENTS.



METHOD OF MEASURING BASE LINE FOR EAST RIVER BRIDGE NO. 4.

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remeasured a year later. The results were 3298.146 ft. and 3298.148 ft., which differ from their mean less than 1 in 3 000 000. Mr. Erlandsen.

The speed attained in these measurements is almost uniformly 400 ft. per hour.

HORACE ANDREWS, M. Am. Soc. C. E. (by letter).—It is to be hoped that this paper may lead our makers of surveying instruments to adopt some of the suggestions of the author. The first change mentioned is an evident improvement. The arrangement suggested has been long used on vertical circles of the higher grades of theodolites. The addition of a sensitive level to the vernier of the plane-table alidade was suggested by Professor Jordan, and a drawing and description of the device are given by him. Professor Jordan also suggests the independent leveling of the alidade at right angles to the line of sight, without re-leveling the plane of the table.* He also refers to the level attached to the vernier as being an advisable adjunct for the tachymeter theodolite.† Again, in referring to the measurement of vertical angles with the theodolite, he calls attention to the need of a level attached to the vernier which can "be brought exactly into play with each individual sight," by means of a suitably attached tangent screw. Professor Jordan also used an arc of large radius, with divisions on celluloid, for reading vertical angles to the nearest minute, the instrument having also a full circle, divided on silver, with two verniers, for more precise determinations. Mr. Andrews.

The second improvement, the increased radius of the arc of the vertical circle, would seem less essential. If a circle of small radius is well divided, and sufficient magnifying power is used, a reading to a half-minute should be easily possible.

However, a larger radius may be used, even as large as that shown in the dotted circle of Fig. 4, Plate I, by attaching an arc of suitable length to the eye-piece and object-glass ends of the telescope. This is done with certain English instruments, and the arrangement has the decided advantage of permitting the use of two verniers. Where only one vernier on the horizontal limb exists, eccentricity errors can be rendered of no account by adopting suitable methods of observing, but with only one vernier to the vertical arc there is no method of procedure in measuring vertical angles whereby the errors of eccentricity may be eliminated. The best that can be done is to determine the amount of the error due to eccentricity and allow for its effect upon the vertical angle measurements. This determination is a tedious process; it is sometimes attempted with sextants, but rarely, if ever, with the vertical circles of transits.

A complete circle possesses many advantages. It could be arranged so as to be removed, with its attached level and verniers; it

* Vermessungskunde. 1877 edition, Vol. 1, p. 647.

† Same, p. 606.

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Mr. Andrews. could then be carried in a small case, and would be safe from injury. Such detachable circles are sometimes used with the higher grades of theodolites. With the complete circle, the vernier need not be double, so as to read plus and minus angles. It can be arranged to read "zenith distances;" that is to say, the reading when horizontal would be 90 degrees. Instead of recording the angle, — 1°, the zenith distance, 91°, would be used, while 89° z. d. would correspond to + 1° of vertical angle. A constant source of mistakes would thus be avoided, while the rapidity of reading would be increased.

Professor J. E. Hilgard's reports of tests of theodolites may be read with profit by anyone interested in examining instrumental errors. In the United States Coast Survey report for 1856, p. 315, he refers to the examination of the parallelism of the vertical axes, and suggests a method of examination, as follows:

"After leveling the instrument as nearly as may be, revolve the circle about the alidade, the latter being held in its position by the hand, and watch the indications of the striding level, which will not change if the two axes are parallel."

The references the author makes to "the motion to which a transit is subject, even under ordinary conditions," as revealed by the sensitive vernier level, would probably indicate a lack of parallelism of the vertical axes. Professor Hilgard's test would reveal such a source of error.

The author refers to the slide rule as if it were solely used for ascertaining the correction depending upon the sin.^{2} of the angle. The 50-cm. slide rule of Porro, which has been used for many years, was also designed for computing the quantities corresponding to $\frac{1}{2}$ sin. $2a$, necessary for obtaining differences of height. By arranging the slide rule in circular form, similar to the "pocket calculator" of W. F. Stanley, the unit of length could be as large as necessary, while the diameter of the circle would still be small enough to render the instrument quite portable.

Perhaps the newly-described Hammer-Fennel's Tachymeter, which does away with all computation, giving both horizontal distance and difference of height, by means of a diagram reflected into the telescope and seen at the same time with the stadia rod; may prove successful, and supersede both vertical circle and slide rule. The novelty and ingenuity of this device should stimulate our instrument makers, and show them that a field for improvement and invention exists.

The writer has often deplored the absence of a circular level for the preliminary leveling up of the ordinary leveling instrument. Such circular levels are very commonly used on foreign instruments, and facilitate very materially the operation of setting up. Judging from the high praise given to the device known as the "*feste Loth*"—the immovable plumb-bob—by the Germans, it would seem worthy of

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introduction by some of our dealers. In windy weather much time is lost with the swinging plumb-bob, and the engineer adopts all sorts of expedients to prevent the loss of certainty in his plumbing; his make-shifts, however, do not always prove successful. The writer has never heard of any device proposed by our instrument makers to facilitate the setting up of a transit, and to supersede the usual, hanging, swinging plumb-bob. Mr. Andrews.

The shifting head used on our transits, has been a very commendable improvement, and has attracted favorable attention in other countries. The writer has an excellent level tripod which is provided with substantial steel points with projecting lugs, to allow the points to be firmly shoved into the ground by the pressure of the foot; and can testify to the value of the device. As made by a well-known firm in this country, the points and lugs are too small and unsubstantial to give good service.

Perhaps too much is sacrificed to convenience and portability with our instruments. The writer has sometimes found it almost impossible to use a wye-level in very windy weather, on account of the deficiency in torsional resistance of the tripod. It is easily possible to make tripods so rigid that the motion from wind will be imperceptible, but such tripods are not quite so comfortable to the shoulders of the one who has to carry the instrument.

Our instrument catalogues are notably defective in their omission of the value of the length of the level divisions, the focal length and aperture of the telescopes, and their magnifying power. The weights of instruments, tripods and boxes are sometimes given in foreign catalogues, and such information is often of interest.

Some instrument makers attach the level divisions to a metal scale which is suspended over the tube. It is claimed that with extremely sensitive levels the etching of the divisions on the tube will cause a gradual change in its radius of curvature. To prevent this, it has been suggested that the divisions should be etched on the inner surface of a glass tube which would surround the level tube and be nearly in contact with it. With either the metal scale or an etched scale on a surrounding tube it would be possible to provide means for effecting a slight longitudinal movement, and thus facilitate what is now a somewhat tedious adjustment.

In speaking of leveling the theodolite, Professor Jordan says:

"This is best made by imagining the level axis changed in place. If, for example, the bubble-center reads 50.4 div., and after reversal reads 47.6 div., then if it is made to read 49.0 div. we can expect it to remain correctly in level at this reading. This new normal point, 49.0, and a few points equi-distant therefrom can be designated on an attached scale."

This "attached scale" already exists, as above remarked, on some of our instruments; it only needs facilities for a slight longitudinal adjustment by a suitable slow-motion screw.

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Mr. Taber. GEORGE AYMAR TABER, Assoc. M. Am. Soc. C. E.—It may be interesting to know how accurately work can be done in measuring distances with an ordinary 100-ft. steel tape, as this problem is one that often has to be met by almost every engineer.

In running the line for the Central Park Tunnel, on Section 7 of the Rapid Transit Railroad of New York City, it was necessary to obtain as accurately as possible the distance across the northwest corner of Central Park from 104th St. and Central Park West to Lenox Ave. and 110th St., a distance of 1 828 ft. This line ran up and down hill, through the trees, bushes and shrubbery of the Park, varying in elevation over a range of about 109 ft.

The measurement was made with an Eddy, 100-ft., steel tape, standardized for a pull of 12 lbs. when suspended at full length in the air at a temperature of 62° Fahr. On account of the steepness of the slopes, it was necessary to measure a considerable portion of the line in short lengths, and the question arose as to what pull to give to the tape for distances of less than 100 ft. To decide this, a base line was laid off 50 and 100 ft. long, on a level sidewalk, by means of a 50-ft. Keuffel and Esser, spring-balance, compensating tape. Then the Eddy tape was applied to this base, to determine the ratio of pull to length of tape. This ratio was found to be practically a constant, so that 6 lbs. pull was necessary for a distance of 50 ft., 3 lbs. for 25 ft., etc., the tape being at all times suspended in the air.

The line across Central Park was measured twice in this manner, once in October, 1900, and again, by a different party, about a month later. Each portion of the line as measured was corrected for temperture at the rate of 0.007 ft. per 100 ft. of length for each 10° of temperature above or below 62° Fahr. The variation between the two measurements was found to be only 0.034 ft., or less than 0.002 ft. per 100 ft. of length. The only outfit used in this work, besides the tape, consisted of a common spring balance, a thermometer, two plumb-bobs and a "Locke's hand-level." The latter was used to sight along the tape from one end toward the other, to be sure that both ends were held at the same elevation.

This method of measuring was found to be so accurate and rapid that it was adopted, and has since been used for all work on Sections 7 and 8.

Mr. Smith. LEONARD S. SMITH, Assoc. M. Am. Soc. C. E. (by letter).—This paper is a most timely and valuable contribution to the literature of the subject. It certainly is to be hoped that, as a result of this paper and its discussion, instrument makers may display renewed interest and enterprise in meeting the demands for improvements. One reason why American engineers have not more generally demanded and secured desired improvements is that, almost invariably, the instrument makers require an extra price for any change, even if the alteration is less expensive in design than the regular detail. In this way

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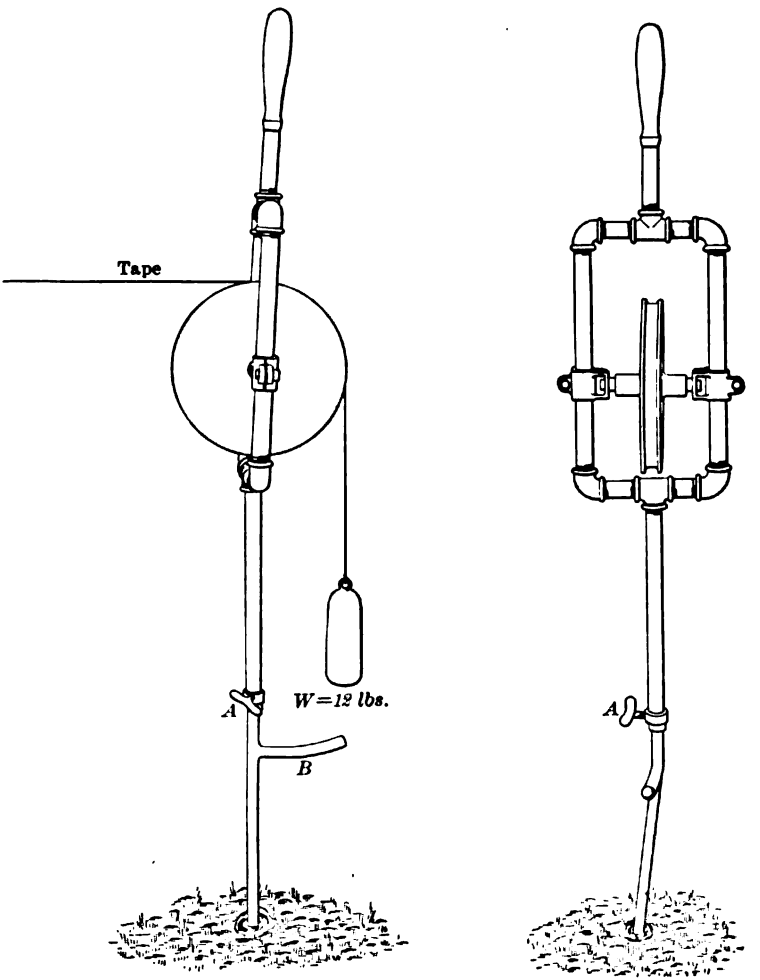

FIG. 3.

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Mr. Smith. the buyer's pocketbook is made to pay the penalty of its owner's progressiveness.

Aside from the natural conservatism to be expected, this non-progressiveness is largely due to the fact that very few instrument makers have ever used their instruments in the field. This precludes them from discovering the shortcomings of their instruments, the first step in the direction of an improved design. Under such circumstances, it is unfortunate that experienced observers have not been more insistent upon improvements in design suggested by their experience.

An improvement in design, in order to be of general use and undoubted merit, should not result in unduly complicating the instrument.

The author's device for stretching a base tape, while doubtless capable of very accurate results, seems clearly to violate this principle, besides being seemingly so unwieldy as to preclude fast work.

The apparatus shown in Fig. 3 was designed and made at the University of Wisconsin, and has been in use by the Students' Trigonometric Survey for six years. It weighs only 9 lbs., and, with the exception of the wheel, is made of 1-in. gas-pipe. The 9-in. wooden wheel turns in bicycle bearings, thus insuring the constancy of the tape tension and the weight, *W*. The lower part of the standard is a solid iron bar telescoping into the gas-pipe above, thereby allowing the height of the wheel to be adjusted and clamped by the wing-clamp, *A*. This solid rod has a projecting arm, *B*, for use in pushing it into the ground.

The apparatus shown in Fig. 4 forms the rear tape-stretcher. With the exception of the iron hook at *D* and the sharpened spike* at *E*, it is composed entirely of light wood, weighing 7 lbs. A long vertical slit is cut in the upright board, in which runs a grooved wooden slide carrying an iron bolt ending in a hook at *C*, and a thread and winged nut at *D*. The tape is attached to the hook, *C*, by means of a short linked chain.

The operator stands on the horizontal base, forcing into the ground the spike at *E*. He then pushes the slide so as to bring the tape tangent to the post carrying the measure, and then turns the nut, *D*, until another observer sees that the zero of the tape and the mark coincide. A signal is then given to the front tapeman, and the new tape length is marked.

In June, 1901, an inexperienced party of students made a forward and backward measurement of a base line 5 700 ft. long, in a total time of 1 hour 45 minutes, using this apparatus and a 300-ft. tape. When all the corrections had been applied, the two measures reduced to:

Forward	5 700.532 ft.
Backward	5 700.539 ft.

* During the past season this spike was omitted, without increasing the difficulty.

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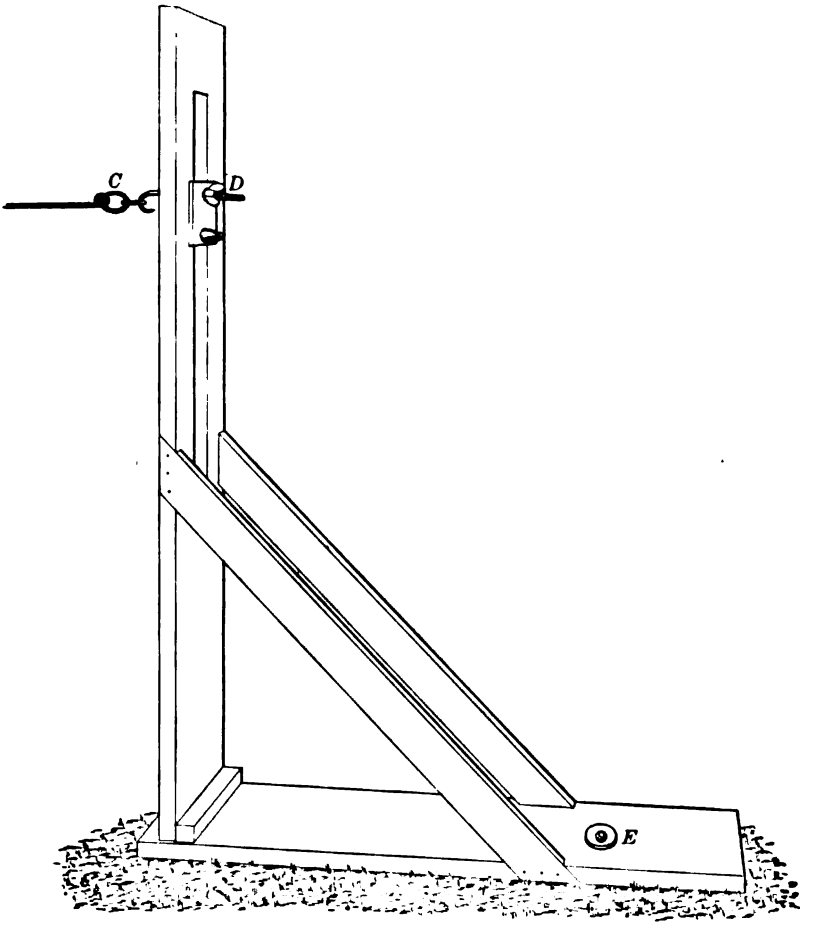

FIG. 4.

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Mr. Smith. This is equivalent to a probable error of the mean of 1 in 2 300 000, as far as the compensating errors are concerned. In June, 1900, a similar party of students, using the same apparatus, got, for a mean length of two measurements of the same base, 5 700.522 ft., which differs from the mean of the 1901 measurement by 1 in 470 000. In each case the second measurement was made at the rate of 1.5 miles per hour, or 2.4 minutes per 300-ft. tape length.

After a critical examination of more than fifty surveying instruments belonging to the topographic engineering department of the University of Wisconsin (representing eight established firms), and including tests on all important physical constants and mechanical defects, the writer is forced to the conclusion that the most common and serious defect in the engineer's transit is the lack of proper coordination of its parts. While the instruments of different makers differ widely, it may be said, in general, that this lack of proper balance is most keenly felt in the following particulars:

(a) The vertical circle being too small;

(b) The omission of a sensitive level from the vertical circle, for the control of its zero;

(c) The use of irregular and non-sensitive telescope level vials.

It is a well-known fact that an error of several feet in the horizontal location of a topographic point would scarcely be discernible on a contour map of ordinary scale, while an equally great error in its vertical location would entirely vitiate the map for its designed purpose. In view of this fact, the time-honored custom of using transits with large horizontal circles, frequently reading to fractions of a minute, results in simply strengthening the work at a point already the strongest, while the custom which sanctions the use of small vertical circles weakens the work at a point where the greatest accuracy is needed. The vertical circle or arc should have a diameter at least equal to that of the horizontal circle.

The usual excuse for the common practice of furnishing transits with smaller vertical than horizontal circles is found in the supposed danger of injuring the larger sized circle. The writer's experience, under the most trying circumstances, incidental to rugged mountain climbing, would indicate that this danger is greatly over-estimated. In fact, if the circle is protected with a cast metal guard, and it always should be thus protected, there can be little danger. The vertical arc, and its ingeniously devised double verniers, as described by the author, is open to the objection that it cannot be read with telescope reversed, for the purpose of eliminating the errors of adjustment, in such problems as, for example, the determination of azimuth by direct solar observation, now so commonly used. Moreover, there is greater danger of injury to the transit provided with a vertical arc when working through underbrush, due to its hooking into the limbs.

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Mr. Smith.

Although the small vertical circles frequently furnished by makers ostensibly read to single minutes, experienced observers are well aware that such accuracy is seldom realized, because of the uncertainty of the reading. Small circles should be provided with permanently mounted magnifying glasses.

A second element which introduces error into the vertical work of a transit is the practice of depending upon cheap non-sensitive plate levels for the control of the vertical-circle zero. If not specified, makers send out plate levels with a sensitiveness between 40 and 360" per $\frac{1}{10}$ in. of level. Now, every topographer knows that after the orienting back sight has been taken and before the fore sight can be taken, especially if many side shots intervene, the instrument will get out of level, a condition indicated by the plate bubbles getting off center. This introduces errors in the vertical circle readings, it may be of equal angular amounts. Nor can the plate levels be re-leveled without introducing an error in the orientation which will affect all subsequent azimuths; but, even if this displacement does not always take place, a sensibility of 40 to 360" is quite inadequate to control the vertical circle. The proper remedy for this evil is found by placing an extra level on the standard, or upon the vertical circle guard, and making it adjustable, so as to read zero when the telescope level is horizontal. If the adjusting screw for this level be placed on the forward or objective side of the instrument, there will be no danger of its being mistaken for the tangent screw on the horizontal axis.

A third needed change in the design of the transit, and one which is certain to improve the accuracy of its vertical work, is found in a rational increase in the sensibility of its telescope level.

Except in very mountainous regions, a properly run topographic traverse by the "transit and stadia method" allows a considerable proportion of the vertical angle readings on turning points to be replaced by level readings, thereby correspondingly reducing the amount of required office work, while, at the same time, insuring a higher degree of accuracy in the resulting levels. This would be notably the case in a combined hydrographic and topographic survey of a river and its adjacent valley. This point was first brought to the writer's attention while making a survey of the Colorado River, for a distance of about 25 miles, near Yuma, Ariz. At this point the river flows in a wide sandy valley of light gradient, so that practically all elevations were taken by level readings.* Although this survey was executed under the most trying conditions of weather, and with an average length of sight of about 300 m., its levels checked on bench-marks determined by accurate spirit leveling within 0.5 ft.

Equally close results have been observed on topographic lines, and

* A light transit, with sensitive level, by Brandis, Son & Company, was used. See, also, for a discussion of such survey, *Journal of the Association of Engineering Societies*, Vol. 14, page 219, by J. L. Van Ornum, M. Am. Soc. C. E.

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Mr. Smith. these lead the writer to the conclusion that a properly devised level on the transit telescope is quite sufficient to secure results under favorable circumstances comparable with those of common Wye-levels, and expressed by the formula: Error, in feet, $= 0.5 \sqrt{\text{distance, in miles}}$, as the highest limit, and, perhaps, twice this error for the lowest limit, the latter being expected only under very unfavorable circumstances as to weather conditions and length of sight.

However, such high-grade work cannot be expected with transits having the non-sensitive and irregularly ground level so frequently furnished the unsuspecting purchaser. After considerable experimental work to determine this point, the writer would suggest a sensibility corresponding to 0.1 in. $= 15''$ of arc for a transit magnifying 20 to 24 diameters.

During the past six years the University of Wisconsin has purchased about forty transits and levels for the use of the rapidly increasing number of students in the Engineering College. Thinking that the specifications, under which these instruments were in most part purchased, would be of interest, they are here given in full. It may be remarked that the custom of purchasing instruments by competitive bids, and of allowing each bidder to write his specification, is obviously unfair to all concerned. Besides making all bids comparable, carefully drawn specifications should insure that the purchaser will receive exactly what he proposes to buy.

Specifications for Complete Engineer's Transits to be Furnished to the University of Wisconsin.

General Conditions.—All constructive details shall be of superior workmanship and design. Weight not essential to the stiffness of the instrument shall be avoided by the standard rib construction. The instrument, without tripod, should weigh not more that 10 lbs. Especial attention shall be given to making extra strong all parts likely to be injured by accident. Only the hardest bell metal and phosphor bronze shall be used for the centers and telescope axis, and soft brass shall be avoided everywhere. The tangent screws and the head of the tripod shall be made of aluminum bronze and provided with German silver spiral springs. Prices shall be f. o. b., Madison, Wis.

Horizontal Circles.—The diameter at the edge of the graduation shall be 5 ins. The graduation shall be to half degrees, on solid silver, and every 10° shall be distinctly numbered from zero to 360° in both directions, the numbers sloping in the direction they are increasing.

The circle shall be provided with two double verniers, *A* and *B*, exactly 180° apart, and reading to single minutes.

The zeros of the verniers shall be placed at an angle of 35° to the line of sight, and shall be provided with ground-glass reflectors. The eccentricity of the vertical axis and of all verniers must be as perfectly eliminated as the highest mechanical skill will allow.

Complete Vertical Circle.—The transit shall be provided with a complete vertical circle, having a diameter of not less than 5.5 ins., graduated on solid silver, and reading to single minutes by one double

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vernier. The vernier shall be provided with a ready and reliable means of adjustment. The vertical circle shall be furnished with a cast metal guard to protect the entire edge of the graduation. An extra level vial (sensibility 0.1 in. = 20″), with suitable means of adjustment, shall be mounted firmly on this guard, so as to control the zeros of the verniers. The horizontal axis shall have a radial clamp and a slow-motion screw. Mr. Smith.

Magnetic Needle.—The instrument shall be provided with the usual compass box and a sensitive and accurate needle about 3½ ins. long.

Telescope.—The objective shall have a clear aperture, not less than 1⅛ ins. in diameter, perfectly achromatic, and with a flat field, suitable for accurate stadia measurements. Both the objective and ocular lenses shall be mounted so that their optical centers lie in the geometrical center of the telescope tube, and provision shall be made to insure that the optical centers shall describe straight lines when focused.

The eye-piece shall be inverting, and magnifying between 20 and 24 diameters, and shall be free from chromatic aberration. It shall also be provided with a prismatic eye-piece having a solar shade.

The regular eye-piece shall have the usual focusing screw, or worm, and a dust guard.

The horizontal axis of the telescope shall be provided with means of adjustment, and shall be fitted with a suitable base for a Saegmuller solar attachment.

Stadia Wires.—Two extra stadia wires, very fine and opaque, shall be furnished and placed equidistant from the middle cross-wire. The total distance apart, of the stadia wires, shall be one one-hundredth of the focal length of the objective.

Telescope Level.—The telescope level shall be ground to a uniform curvature of 0.1 in. = 20 seconds. The graduations shall extend over the entire exposed length, and shall be numbered both ways from zero at the center.

Plate Levels.—Both plate levels shall be accurately fixed to the transit, with the most approved and stable provision for their adjustment. Both levels shall be graduated, and their curvature shall be 0.1 in. = 30 seconds.

Clamps.—The heads of all clamps shall be one-third smaller than the usual pattern, to prevent injury from inexperienced hands.

Finish.—The telescope, telescope level, vertical circle guard, and standards shall have the so-called "cloth finish," the remainder shall have the bright finish.

Tripod.—The tripod shall be of the split-leg pattern, and provided with a shifting center. Its weight shall not exceed 7 lbs. The steel shoe of the tripod shall be provided with a shoulder for forcing it into the ground.

Box.—The instrument shall be furnished in a neat hardwood box with lock, together with a sun-shade, reflector, magnifying glass, plumb-bob, wrench, adjusting pins, and silk or waterproof bag. The box shall have a leather strap, and rubber cushion on the bottom.

That there may be no doubt in the reader's mind regarding the need of definite specifications similar to the foregoing, it may be said that, before this plan was adopted at the university, the instruments were seriously lacking in important details. For example, one of the

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Mr. Smith. oldest and best known American firms furnished a transit the telescope level of which had a sensibility of 0.1 in. = 72″ (about suitable for a hand-level), and having plate bubbles with a sensibility of 0.1 in. = 360 seconds. Though the horizontal circle of this instrument is nearly 7 ins. in diameter, the vertical circle furnished has a diameter of only 4 ins. The results obtained by this instrument have shown a uniformly larger error than that shown by the standard instruments purchased under these specifications.

A comparison of the plate bubbles of ten similar instruments purchased simply by catalogue number shows that not only do different firms use widely different levels on transits intended for the same grade of work, but even the same firm at different times; the range on plate levels being from 40 to 360″ and on telescope levels from 8 to 72″ per 0.1 in. The conclusion, that makers frequently use whatever they happen to have in stock, quite regardless of suitability, seem warranted by the facts. Some will not fail to urge the impossibility of buying a transit at a reasonable price under such rigid specifications as suggested herein. In answer, it can be said that the average price paid for a transit under these specifications has been considerably less than formerly, without specifications. During the present year, the lowest bid, from an American maker, was $175 per transit. The university owns several transits made by this maker, and several field tests have shown them to be the equal of any, both in accuracy and durability.

Surely the specifications of so important a part of the field engineer's outfit as his transit should require at least as much thought and care as he gives to the purchase of his $35 bicycle. That this care and caution should be often entirely lacking is alike discreditable to his professional knowledge and business skill.

Mr. Webb. WALTER LORING WEBB, Assoc. M. Am. Soc. C. E. (by letter.)—The writer feels gratified at the interest in the subject which has evidently been aroused by the paper. A large part of the discussion consists of the description of similar "devices for increasing the accuracy or rapidity of surveying operations," and needs no comment. The discussion, however, has developed the need for some further explanations. The plane-table alidade described in the paper was an old-style instrument, having originally a fixed vertical arc on the standard and a vernier arm rigidly attached to the telescope axis. The accuracy of the vertical angle depended on the leveling of the plane-table board. To have adopted the essential features of the United States Geological Survey alidade or the United States Coast and Geodetic Survey alidade (Figs. 1 and 2, Plate IV) would have required an unprofitable reconstruction of an old instrument. In fairness to the design, however, it should be pointed out that, as the amplitude of motion of the vernier arc is very small, it was possible to place the vernier level between the standards, where it is well protected from accidental blows,

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and the writer believes that the design shown is as free from probable disturbances of adjustments as some of the others, although the newer types have unquestioned advantages; and, after all, it was the principle of a quickly adjustable vernier to which the writer wished to call attention, and, at the time that the University of Pennsylvania plane-table alidade was thus modified (several years ago) according to the writer's plans, he had never seen even an illustration of a similar device. Mr. Webb.

There is no need of adopting a special method of numbering the figures on the two verniers on a shortened vertical arc, as a means of distinguishing positive and negative vertical angles. If the line of collimation is so nearly level that any doubt exists as to the direction of its slope, a mere glance at the large bubble underneath the telescope will instantly answer the question beyond a possibility of confusion.

It is possible that greater accuracy is obtainable when a long striding level is used on a plane-table telescope and the telescope is first leveled for each "shot;" but it should be remembered that, after the telescope has been leveled, it must be raised or lowered until the horizontal cross-wire is at the proper reading on the rod. While taking that reading, there is nothing to indicate whether the table may have been jarred while the telescope was being adjusted, or whether some slight knock which it received had any appreciable effect in altering its adjustment. By the other plan, the vernier level will always indicate the leveling of the table. By one method the telescope must actually be brought level between every shot, besides reading the vernier. By the other method no time is necessarily lost. The vernier level will at once indicate whether a true vertical angle may be obtained, regardless of the precise leveling of the table itself. Frequently, several shots may be taken without re-leveling, and even, when necessary, a mere touch of the adjusting screw, made very quickly, will suffice to restore the adjustment. Practical experience with this device has shown this to be the case. The loss in time due to re-leveling for every shot, therefore, is much more than that admitted by Mr. Matthes on page 109.

The stadia slide rule was of course designed to obtain differences of elevation as a function of $\frac{1}{2}$ sin. 2α, and that is its principal use. The comment regarding sin.$^2\alpha$, in the paper, was made to show the desirability of obtaining a function of sin.$^2\alpha$, rather than a function of cos.$^2\alpha$, which is the function obtained on most other stadia slide rules.

The writer admits that a greater range of vertical adjustment of the tape tension apparatus would have been desirable. Experience has shown that the range for lateral and longitudinal adjustment is ample, and that it simply requires more care in setting up the apparatus in its proper place to get it within the range of the vertical adjustment.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 920.

THERMO-ELECTRIC MEASUREMENT OF STRESS.*

By C. A. P. Turner, M. Am. Soc. C. E.

With Discussion by C. A. P. Turner, M. Am. Soc. C. E.

In undertaking the labor of preparing a paper on this subject, the writer was influenced by the hope that a full description of the methods used, combined with an exhibition of the results obtained under somewhat unfavorable circumstances, together with a few remarks regarding the errors involved and the probable accuracy attainable under more favorable conditions, might lead to further investigation along this line, supplying reliable data as to the fiber stress in columns, the extent of stress induced in members and details of structures by the impact and vibration under rolling loads, together with some insight as to the magnitude of secondary stress, resulting from the character of the details used.

Further, as will be noted later in the paper, this method appears to throw some light on the limits at which molecular fatigue commences to appear, confirming Bauschinger's suggestion that in Wohler's experiments, in which bars broke nominally below the elastic limit of the material, there is reason for concluding that the loads were really greater than the true elastic limits of the bars.

* Presented at the meeting of March 19th, 1902.

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Methods of Measurement.—In the measurement of stress by thermo-electric means, two radically different methods may be used:

A.—By relative measurement of the temperature change in the bar when the load is applied or removed. This method is applicable only while the load is being applied or removed, and cannot be used at any time thereafter, as the thermal change in the bar is gradually dissipated by radiation.

B.—By measurements indicating the extent of the temporary change in the thermo-electric intensity of the bar, due to the stress applied. This change subsists as long as the constraining force is kept applied, and hence the method is applicable to the measurement of either live or dead-load stresses.

In the work done thus far by the writer, measurements by the first method have been reduced to a basis approaching scientific accuracy, while sufficient work only has been done by the second method to indicate its practicability with suitable apparatus, and give reason for thinking that the same amount of time spent in improving the method of operation will place it on nearly an equal footing with direct thermal measurement.

Returning now to the first method: The general law upon which the thermal measurement of stress depends may be stated as follows: Any substance which expands upon being heated absorbs heat (*i. e.*, grows cooler) upon being stretched, and *vice versa*.

Sir William Thomson (*Lord* Kelvin) has developed* the mathematical theory of the thermo-elastic properties of matter, and has demonstrated the following formula as applicable to the phenomena in question:

$$H = \frac{t\,p\,e}{J\,S\,W};$$

where H = thermal increase, in degrees Centigrade;

t = temperature, Centigrade, from absolute zero;

J = the mechanical equivalent of the thermal unit;

p = pressure applied, in pounds, positive if compression and negative if tension;

e = the longitudinal expansion per degree Centigrade;

S = the specific heat;

and W = the weight, in pounds per foot of length of bar.

* *Quarterly Mathematical Journal*, April, 1855; reprinted in the *Philosophical Magazine*, 1878.

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This change in temperature, as would be inferred from the undulatory theory of heat, takes place instantaneously, with the application of the pressure, or load, p, and is subject to the usual laws of radiation. Since the temperature change is very small, Newton's Law of Cooling may be taken to apply exactly, namely: The amount of heat gained or lost by a body in a given time is proportional to the difference in temperature between the body and the surrounding medium. Hence, if two loads, of different magnitudes, are gradually applied in equal periods of time, the temperature changes in the bar for the respective loads will be strictly in proportion to the loads, notwithstanding the radiation going on while the loads are being applied.

The formula, $H = \frac{t\,p\,e}{J\,S\,W}$, is rigidly exact for any solid only for an infinitesimal increment in p; but, with steel, the variation within the elastic limits would be extremely small, and may be taken as exactly proportional to the pressure or load applied, and, as we are interested only in relative measurements, no error is involved.

As H depends on t, the temperature of the bar at the time the load is applied, this temperature should be noted, if readings taken at different temperatures are to be compared. Thus, if the value of absolute zero be taken at — 276.7° Cent., and one set of measurements is taken at 23.3° Cent., and another at 24.3° Cent., the latter, for the same loads, should be $\frac{1}{8}$ of 1% greater, if no error has been made in the work. As t would probably not vary greatly, this percentage per degree Centigrade is sufficiently accurate for most purposes.

The magnitude of the temperature change, H, may be best illustrated by a few figures. Take the temperature of a bar at 19.3° Cent., p at 1 lb., and the area of the bar at 1 sq. in., and we have, for mild steel, $H = \frac{(276.7 + 19.3) \times 1 \times 0.00001142}{1400 \times 0.11 \times 3.34} = 0.00000657°$ Cent., the temperature change for a change in stress of 1 lb. per square inch.

To one not familiar with thermo-electric work, this temperature change per pound per square inch would seem to be much too small to admit of measurement, but this is by no means the case, if, in connection with the thermo-pile, there is used a galvanometer as sensitive as a Thomson astatic. The range through which we may measure, however, will be correspondingly small, as may be best illustrated by the result of the following test, by Mr. McIntyre and the writer, using a fairly sensitive adjustment of an Elliott Brothers' instrument.

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On application of a load of about 30 lbs. per square inch on the bar, a deflection (scale reading) of more than 60 cm. was noted (a deflection which is nearly the maximum which may be read). This is at the rate of 2 cm. per pound per square inch; and, as 0.3 mm. is a very appreciable deflection, we would readily note a change in temperature in the bar of one ten-millionth part of 1° Cent., caused by the application of a load of about ½ oz. per square inch, and this is by no means the limit.

For practical purposes, however, we would of course desire to take the measurements through a range of several thousand pounds per square inch, and an adjustment which would give a deflection of 2 cm. per 1 000 lbs. would be suitable. Now, with such an adjustment we would endeavor to read to 0.1 mm., which, with this adjustment, would correspond to about 5 lbs. per square inch, giving a temperature change of a little more than three-hundred thousandths of 1° Cent. This being the probable error of reading with such an adjustment, the percentage of error in the reading would decrease, the greater the deflection, being ± 0.5% for 1 000 lbs., and ± 0.1% for 5 000 lbs., per square inch, etc.

Apparatus.—The necessary apparatus will now be described in detail. It consists of a thermo-pile, or junction, attached to and insulated from the bar, in circuit with a suitable galvanometer, provided with the usual reading scale and telescope for reading the deflections. In the experiments given, the load was applied to the bar by an Olsen testing machine, and weighed off as carefully as possible. A wire index was attached to the end of the scale beam, and a scale was fastened to the standard, in order to enable the operator to weigh, as accurately as possible, the load applied.

In the writer's early work in this line the galvanometer used was a Thomson astatic, manufactured by Elliott Brothers. While this is an excellent instrument, it is more sensitive than necessary, and has the serious disadvantage of being affected by the slightest external magnetic or electrical disturbance. Hence, in the later work, a D'Arsonval type of galvanometer was used, which, from the character of its construction, is practically uninfluenced by conditions which would render a Thomson useless.

The D'Arsonval type consists of a permanent horse-shoe magnet, between the poles of which is suspended by a bronze fiber a coil of

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copper wire. The current passes down the fiber and through the coil, and the coil tends to place itself at right angles to the lines of force of the magnetic field. As the coil turns it is resisted by the torsion of the fiber.

Assuming the field to be constant, the torsional resistance of the fiber is, within the elastic limits, in proportion to the deflection angle, $\triangle$, while the lever arm of the deflecting force is cosine $\triangle$. Hence, the current strength is in proportion to the angle, $\triangle$, divided by the cosine $\triangle$, while the readings (the zero being at the center of the scale) are to each other as the tangents of twice the deflection angles.

Were the field strictly uniform, the mathematical corrections to the readings could be readily made and applied. The assumption, however, that we have a constant field may be far from exact, and should be investigated experimentally.

Using a standard cell, giving a constant current, we may, by the use of suitable resistance boxes, arrange to shunt but a small fraction of the current through the galvanometer, and, by varying the resistance, may vary the current passing through the main circuit; then, placing a standard ammeter in the main circuit, we can measure the current, and, by comparing the galvanometer deflections with the readings for current strength, we may determine the galvanometer corrections.

As we cannot read the ammeter with the same precision that we can read the galvanometer, it is better to use a large tangent galvanometer in place of an ammeter, where this type of instrument would not be disturbed by earth currents, caused, as was the case in the laboratory where the writer worked, by the proximity of electric street-railway lines, which prevented a more accurate investigation of the instruments used.

The temperature correction, in the comparison of measurements made at different temperatures, has been noted, but it should also be noted that any change in temperature changes the resistance of the copper wires in the circuit and coil, and a correction should be made for this, where the accuracy of the work would warrant such a refinement.

The most difficult correction to deal with satisfactorily is that of time or radiation, since the measurements depend on:

1. The temperature change in the bar, which is effected by radiation;

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2. On the conduction of this temperature change to the end of the pile next to the bar, through the insulating material;

3. On the time required for the galvanometer to register the full amount of deflection.

In the experiments presented in this paper, the time recorded is that from the time of starting to apply the load until the reading was taken, and an attempt was made to determine the percentage of error involved for a given percentage of increase or decrease in the time noted for each application of the load. One of the difficulties of doing this accurately lies in the fact that the galvanometer reading is nearly stationary for a second or two at or close to the maximum, and it is difficult to determine the time exactly.

Where the time of application of the load is short, say, 25 to 35 or 40 seconds, a considerable difference in the observed time appeared to make little or no appreciable difference; while, where the application of the load was made at a much slower rate, say, 1 minute to 1 minute, 20 seconds, a variation of 10 or 12 seconds appeared to make a quite regular, though small, percentage of difference in the reading for a number of experimental determinations.

One feature of the apparatus, which was given much thought and attention, was the proper method of attaching the junction, and the method finally used was somewhat novel.

In the writer's early work, difficulty was experienced with the antimony-bismuth pile by its getting in contact with the bar, and short-circuiting, thus making the readings unreliable; then, if he tried to compare different readings, the distance or air space between the end of the pile and the bar might have varied, and the results could not be compared with a high degree of accuracy.

Again, air being a poor conductor of heat, the results were not altogether satisfactory from this standpoint. To obviate this the bar was given a coat of shellac, and the thermo-pile applied. By this means much better results were secured. It appeared, however, that a little difference in the thickness of the coating of shellac and in the time which it had been allowed to dry made some little difference in its conductivity, and an attempt was next made to make the junction with gum-arabic in place of shellac, using a very thin piece of paper coated on each side, moistened, and applied to the bar, and the thermo-pile pressed against the other side.

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This not proving wholly satisfactory, rubber cement was used for the junction. It was found that this could be applied quickly, would dry in a few minutes, and, as it gave a thin uniform coating, measurements with different junctions could be compared with a fair degree of accuracy. The thermo-pile was held in place on the bar by elastic bands.

In addition to the cap over the outer end of the pile, to protect it from air currents, the head of the machine was wrapped with heavy asbestos board and carpeting, and, in order to keep note of any temperature change that might occur, a thermometer was inserted through the covering, and read at intervals, while a thermometer was placed outside in order to keep note of temperature changes in the room.

The laboratory where the experiments were performed, while admirably equipped for the purposes intended, presented few advantages for the work in hand except the convenience of the Olsen testing machine. The room itself was subject to air currents and considerable temperature changes, and the galvanometer, instead of having a specially prepared masonry base on which to stand, was set up on the plank floor. A further disadvantage of earth currents, due to electric street-railway traffic, has been referred to.

Fig. 1, Plate VIII, is a side view of the large testing machine used, with the covering of the head not yet applied. It shows a thermo-pile attached to each side of the bar. The wire index and scale, to assist in weighing accurately, may be seen at the right.

Fig. 2, Plate VIII, shows the machine in the background, with the head covered, and in the foreground the two galvanometers and the telescopes with scales for reading the deflections. The large galvanometer at the left is a Queen instrument, the field of which was found to be very nearly constant, the readings requiring but slight corrections. The instrument to the right is a small Knott galvanometer. The field magnet of this consists of three steel horse-shoe magnets a small distance apart, and, as would be expected, the field was not exactly constant, and further, the suspending fiber, being very short, was more easily over-strained, and was less reliable. Hence, the results with the small galvanometer are not nearly so reliable as those obtained with the Queen instrument, and its use, where two sets of readings were taken, was more as a check to determine whether

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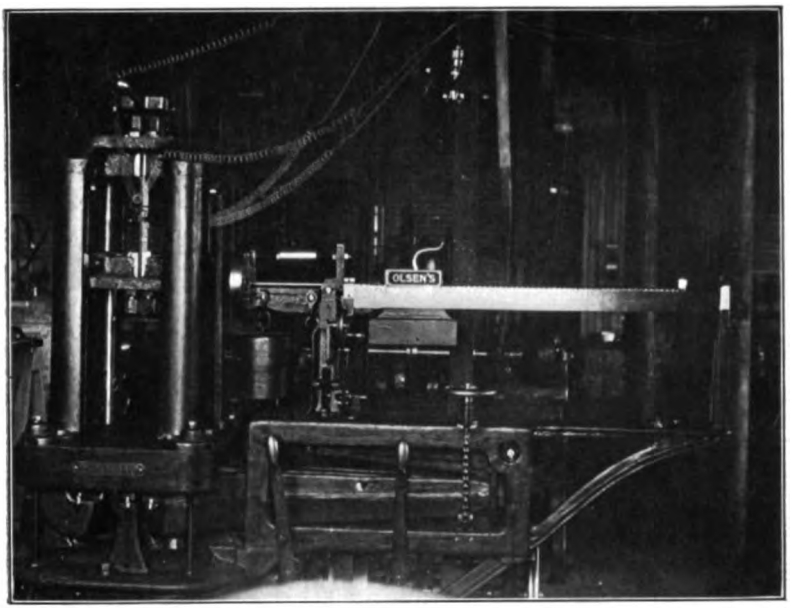

FIG. 1.—LARGE OLSEN TESTING MACHINE.

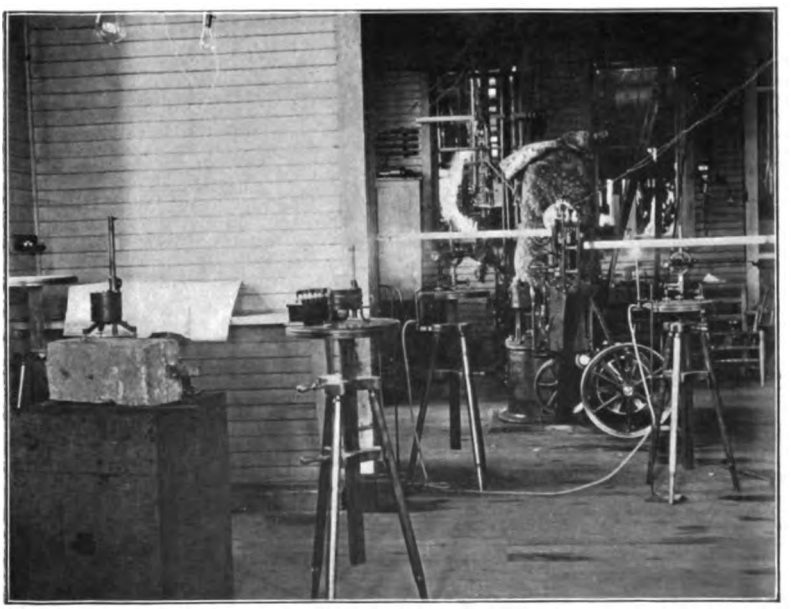

FIG. 2.—GALVANOMETERS, SCALES, ETC.

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the bar was in bending or not. The importance of this can better be understood by bearing in mind the fact that the readings represent the mean fiber stress on the bar only in so far as the load is uniformly distributed over the section of the bar; thus, if the bar were slightly bent the tension would be considerably greater on one side than on the other, by the amount of bending in the bar, and this difference would be indicated by comparing the readings of the two galvanometers, and would be particularly noticeable as the bar straightened out, under increased load. In other words, the galvanometer readings indicate the average fiber stress immediately under the area covered by the thermo-pile. Where the chucks are not exactly in line with the bar a certain amount of bending may result from this cause. Some of the bars were planed and some were rough, as is noted in the description.

Plate IX shows the small Olsen testing machine.

For the benefit of any who may be interested in experimental work in this line, a statement of the cost of the necessary apparatus may be of interest, and the following figures are approximately correct:

Queen D'Arsonval galvanometer	$40.00
Telescope and scale	35.00
Stand	5.00
Antimony-bismuth thermo-pile, 36 couples, about	30.00
Wire	0.50
Rubber cement	0.25
Benzine	0.25
Total	$111.00

For work such as the writer has endeavored to do, duplicate apparatus would be found more desirable than the use of galvanometers and piles of different size and make. Thus, the large thermo-pile noted had 49 couples, the small thermo-pile had 25.

The experiments given cover only tensile stresses. The writer had made, previously, a number of experiments on short bars in compression, sufficient to determine the general character of the thermal stress curve in compression, but, as compressive tests are far more difficult to make accurately than tensile tests, and the writer's time was limited, none was attempted for this paper.

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Thermal Stress Curve.—The mathematical law of change in temperature with the application of compressive or tensile stress has been noted, and attention will now be called to the limits within which this law holds good.

In soft steel of good quality, ranging from 50 000 to 58 000 lbs. per square inch, ultimate strength, this limit appears to be from 18 000 to 23 000 lbs. per square inch. When the pull exceeds this limit the temperature change is no longer proportional to the pull or load applied. The curve deviates from a straight line, at first slowly, then more and more rapidly, as the yield point is approached, when heat is rapidly generated.

For medium steel from 60 000 to 70 000 lbs. ultimate strength, this thermal limit of proportionality is considerably higher, from 23 000 to about 27 000 lbs. per square inch, and when the stress exceeds this limit the curve appears to deviate more rapidly from the straight line than is the case with the softer and tougher metal. When the yield point is reached, or approached closely, heat is generated more rapidly than is the case with the softer metal.

Under compression, the metal under stress grows slowly and regularly warmer, in proportion to the pressure applied, until approximately in the vicinity of these limits, when the temperature increase is at a considerably more rapid rate, and, as the yield point is approached, there is, as was the case in the tensile test, a rapid rise in temperature.

Fig. 1 shows typical thermal stress curves in tension and compression.

Relation of the Thermal Limit of Proportionality to Fatigue.—Referring to Fig. 1, it will be noted that the thermal limit of proportionality is lower than what is usually considered the true, primitive elastic limit of the metal, and the question as to the reason naturally arises; and, further, what is going on within the metal and between the molecular groups that caused the deviation of the thermal curve from a straight line?

Evidently, were there any cause producing heat, it would offset the cooling effect of stretching the metal. Now, it is known that as the yield point is reached there is a very considerable amount of heat generated, accompanying the breaking down of the structure of the bar and the flow of the metal, and it may well be inquired whether this

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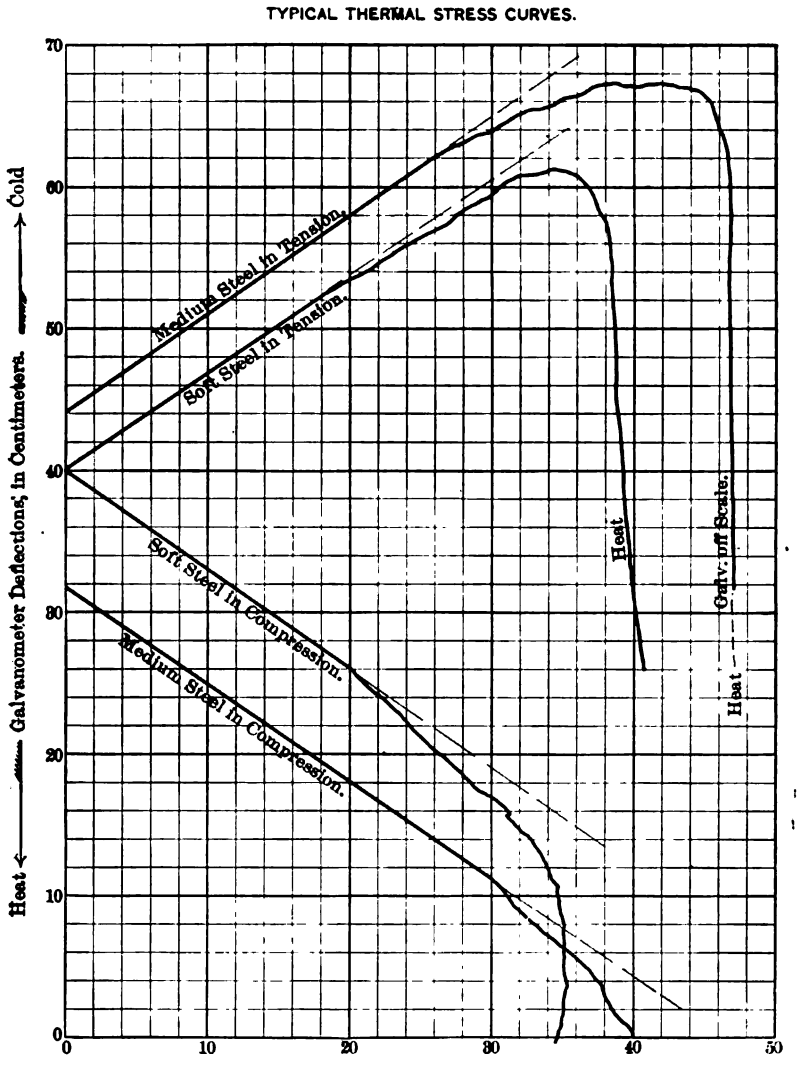

FIG. 1.

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limit of thermal proportionality marks the incipient stage of this change in structure under stress. If this be the case, then this limit should agree closely with Wohler's range for unlimited repetition for alternating stresses, which appears to be the case.

Further, if the deviation of the thermal curve from the straight line (indicating the heat generated by non-elastic, internal friction) be taken as a measure of the injurious effect of the range of stress above this limit, then it should be inferred that the harder metal would show, in proportion to its strength, inferior endurance as compared with the softer metal, under repetition of all stress of sufficient magnitude to induce fatigue, and the accuracy of the inference seems to be substantiated by the comparison of all the records of endurance tests on the two classes of metal.

If there is any such internal friction as has been assumed accompanying the strain of the metal beyond the thermal limit of proportionality, but within the apparent limit of the proportionality of shape, it must be conceived that there has been an expenditure of energy in overcoming the internal resistance of the breaking up of molecular groups, and friction incident thereto. Granting this, its cumulative effect would be in evidence in the increase of the period and the rate of subsidence of the elastic vibrations of the specimen. Thus, in Thomson's experiments, two similar and equal pieces of copper wire were put up, about April 26th, hanging with equal and similar lead weights, similarly fixed by soldering. No. 2 was more frequently vibrated than No. 1, but no comparison was made until May 15th; then No. 1 subsided from 20, initial range, to 10, in 97 vibrations, while No. 2 gave the same subsidence in 77 vibrations. During the greater part of May 16th and 17th, No. 2 was kept vibrating, and No. 1 quiescent, and experiments were made, with the following results:

No. 1 subsided from	20,	initial range,	to	10,	in	99	vibrations	of	2.4	seconds.
No. 1	"	20,	"	"	10,	"	98	"	"	2.4	"
No. 1	"	20,	"	"	10,	"	98	"	"	2.4	"
No. 2	"	20,	"	"	10.	"	58	"	"	2.45	"
No. 2	"	20,	"	"	10,	"	60	"	"	2.45	"
No. 2	"	20,	"	"	10,	"	57	"	"	2.45	"
No. 2	"	20,	"	"	10,	"	60	"	"	2·45	"

This series of experiments, from which the foregoing has been quoted, indicates elastic fatigue for the considerable range of stress through which they were conducted, this range being, of course, within

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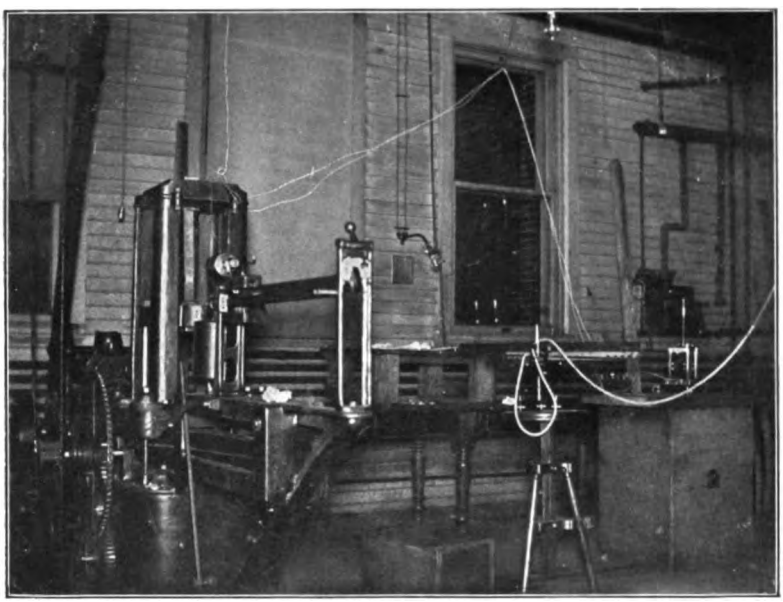

SMALL OLSEN TESTING MACHINE.

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the apparent limit of elasticity of shape. Accordingly, it was supposed that this elastic fatigue occurred for all stresses within the apparent limit of elasticity of shape until a similar and more extensive series of experiments, conducted by H. Tomlinson, showed that no such fatigue of elasticity is felt in steel, iron, or copper, providing the stress does not exceed a certain limit.* This limit is probably identical with the thermal limit of proportionality.

What, then, is the nature of fatigue? Johnson, in his "Materials of Construction," attempts to account for it by micro-flaws in the material. If their effect is the prime cause of ultimate fracture in all cases, we should look for their influence in the elastic fatigue of Thomson's experiments, but here we are met with an unavoidable difficulty in the condition that the elastic fatigue noted depended on the immediately previous molecular condition, whether of quiescence or recurring changes of shape.

Hence, if the speculation were to be continued further, it might, perhaps, well be along the lines of Maxwell's theory of viscosity, by supposing a temporary but progressive increase in the number of unstable configurations, or groups, of molecules under the internal friction noted, with a consequent temporary diminution of tenacity, resulting in the increased period of vibration noted, or in endurance tests in the ultimate development of temporary spots or planes of weakness, wherever these unstable groups chance to be a maximum, and final fracture corresponding to that of brittle material, in the absence of plastic drawing out.

Leaving this interesting field of speculation to others, the question which presents itself to us as engineers is this: If the fact can be established that, for the material we are using, there is a well-defined range of almost perfect elasticity, and beyond which there is a considerable, in fact, nearly equal, range of imperfect elasticity, before reaching the limits of apparent elasticity of shape, are we justified in considering the results of fatigue tests, under ranges of stress extending well into the latter range, as in anywise applicable to range of stress well within the former limit? In view of Bauschinger's work, the writer would hold that we are not.

Even though we discard making a special allowance for the effect of the range of stress of any kind, taking care of the impact and vibration

* *Philosophical Transactions*, Royal Society, 1886.

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due to live load by a rational allowance, we have yet to consider somewhat the case of range of stress from tension to compression. Owing to the fact that the compression formulas give no direct data as regards the fiber stress in columns, we must needs allow an extra margin of safety to cover our ignorance in this respect, until we shall have developed our column formulas so that we know what the actual compressive fiber stress is, since Wohler's work shows beyond question the rapid injury resulting from range of stress from plus to minus when it exceeds well-defined limits.

Such an allowance, however, if our position be correct, should be based more rationally on the length of the column and the amount of the compressive stress involved, rather than upon the total of compression and tension, as is usually done.

Thus far in the consideration of fatigue tests it has been supposed that the stresses applied in the Wohler experiments were really or approximately those which he intended they should be—limited by the high degree of ingenuity displayed in the design of his testing apparatus to the approximate equivalent of a gradually applied load in the absence of shock.

Now, it is well known that the effect of a suddenly applied load in producing deflection is greater than that due to the same load at rest, and it may be demonstrated mathematically that were the load imposed with complete suddenness, or instantaneously and without shock, its effect is to produce a momentary deflection and strain twice that of the same load at rest.

Hence, Mr. Fidler argues that it is permissible to doubt whether the apparatus used by Herr Wohler did in fact succeed in limiting the internal stresses to the intended amount, without exceeding it, and whether the bars were not subjected to the action of dynamic stresses every time the load was applied, which was done about four times a minute. That this was the case to a certain extent can hardly be doubted, but what the limit of the dynamic effect was, is the question to be decided, if Wohler's results are to be interpreted on this basis.

A careful study of the Wohler machines would probably lead the majority of engineers to select at once that for repeated bending in opposite directions, as the one most likely to cause the maximum dynamic stresses, and perhaps the maximum estimate of this effect, in view of the slow speed of the machine, four revolutions per minute,

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would be under 20 to 30% of the load on the cantilever end of the specimen.

Mr. Fidler's estimate for this, however, would be six to ten times that which we have made, by considering it possible that elastic vibrations induced might bring the internal stress up to the maximum effect of a suddenly applied load, which, in this case, would be three times the load, since the range of stress is twice the load.

The following is his table for the minimum breaking load, a, by the dynamic theory:

1. Steady load, no variation.....$a = t$*.
2. Load varying from 0 to u.....$a = u = \frac{t}{2}$
3. " " " $-v$ to $+v$.....$a = v = \frac{t}{3}$

If we subject a bar to a suddenly applied load, the external work of the load is its weight times the deformation of the bar, which must be balanced by the internal work, which is the product of the mean stress times the elastic deformation: Hence, the deflection, or deformation, for the suddenly applied load, will be twice that caused by the same load at rest, since the mean internal stress is only half the maximum. If, however, this dynamic stress exceeds the yield point, part of the internal work will be performed in plastic deformation, resulting in raising the yield point, just as with a steadily applied load. Hence, in ductile material, a suddenly applied load greater than half the ultimate strength would have to be applied a number of times to produce rupture, and on this basis Mr. Fidler undertakes to account for the endurance of specimens in the Wohler experiments where the dynamic stresses would be (if figured on his proposed basis) considerably greater than the nominal strength of the material.

Now, the yield point in mild steel is approximately 60% of the ultimate strength, and, if we limit the range, plus and minus, of the alternate stresses in the Wohler machine to something less than one-third of the ultimate strength, the endurance of the piece seems practically indefinite, and the yield point has not been raised, as no plastic deformation occurs, although the dynamic stresses by Mr. Fidler's method of computation have exceeded the yield point by between 40 and 50 per cent. Since the conditions of application of the load

* t = ultimate strength, static test.

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have been the same for the lower and higher intensities of stress, the same line of reasoning advanced to support the probable accuracy of the dynamic theory may be used with far better advantage to prove its improbability.

It has been argued that, in a short test piece under direct stress, the amplitude of the elastic vibrations under these dynamic stresses would be so small as to escape observation. This argument would have indeed some weight where the stresses are direct and the test piece short, but as similar fatigue occurs under bending, where the length of the specimen would insure so considerable an amplitude of vibration as to render their detection certain if these stresses were of anything like the magnitude supposed, the argument would seem to be invalid.

The final difficulty with this hypothesis will now be considered. If the dynamic effect of a load imposed instantaneously and without shock, in producing deflection and stress, may be readily proved mathematically to be momentarily twice that of the same load at rest, it by no means follows that we can do so if the time of application is extended through the interval of 15 seconds.

In fact, it would appear that this final absurdity has caused not a few engineers to discard the hypothesis, in framing their later specifications, and to make an allowance for the dynamic effect of loads on the more rational consideration of the length of span, or that portion of it which, when loaded, causes the maximum stress in the member, together with the character of the traffic, or the loading to which the structure is subjected.

Variations of the Thermal Stress Curve, under Varied Loading of the Specimen.

As will be observed by referring to the value of the temperature change for a change in stress of 1 lb. per square inch, the maximum cooling effect of stretching a bar up to nearly the yield point will be but little over one-fifth of 1° Cent., an amount too small to be appreciable by touching the bar with the hand. When the yield point is reached, however, the bar warms up very perceptibly, and this point in the curve may for convenience be termed the heat limit. This heat limit varies with the yield point, but the limit of proportionality of thermal change follows a different law. It may be readily lowered, but the writer has not apparently been successful in raising it above

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its normal value. In fact, work thus far indicates, under treatment which is usually supposed to raise the limit of proportionality of shape, that, while the deviation of the thermal curve from a straight line for greater loads is much less marked, it commences at a much lower point.

The following tests are attached as an example of the accuracy of thermal measurement of stress, rather than to demonstrate the foregoing general statements, for the reason that, while some of the tests appended were not made with sufficient care or under sufficiently favorable conditions to be of scientific value in determining the points referred to, they all indicate the value of the method as a means of measuring stress, and the best of them are probably within the limit of error involved in weighing off the load by the testing machine used. In fact, where the Queen galvanometer was used, the writer has no hesitation in stating his conviction that the galvanometer readings were far more accurate than it was possible to weigh the loads with the testing machine used. Especially in the tests where the Knott galvanometer was used, where the first scale reading differs materially from 30, the galvanometer correction is considerably in error for the larger deflections.

Again, the chucks of the large testing machine were such that the bar could not readily adjust itself to the line of pull, involving some little error in bending, which appears in some of the tests.

The heat from the lights used in the reading of the scale affected the pile on the side next to them to a small extent, which was noticeable in the change in the zero or first reading. Where a pile was used on each side of the bar it will be noticed that frequently one reading increased while the other decreased, showing bending, and if the thermo-piles had been of the same size these could have been correctly averaged, thus getting a true mean result.

In Experiment 5, a rough bar was slightly curved on one side, and straight on the other. The effect is quite marked, as may be noted by the readings.

Another error which is involved in making the corrections lies in the fact that the radiation correction was determined while using rubber cement insulation for the junction, and was applied to the experiments made while using shellac insulation, for which it was probably somewhat too great.

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In all cases the loads were applied by the hand-wheel of the testing machine, with as great regularity as possible. The percentage of error in weighing off the load was smaller for the smaller loads, for the reason that the weighing could not be done until after the galvanometer reading had been taken, as any vibration of the scale-beam would affect the reading.

It was noticed that the scale-beam would gradually sag after the application of a load of approximately 20 000 lbs., and the question arose as to whether this was due to squeezing the lubricant from between the teeth of the spur gears, and from between the thread and nut of the screws, or whether it indicated a certain amount of the actual stretch of the metal. If the latter, then the substitution of a bar of twice the size should change the position of this limit in weighing, but as the behavior of the scale-beam seemed to be the same, regardless of the size of the bar, it seemed evident that the first explanation was the correct one. The error involved would be this: The load weighed off would be slightly smaller than it should be, and hence the galvanometer deflection per 1 000 lbs. would be a trifle greater than it ought to be.

Experiment 1.

Evening, August 6th, 1901.

Bar No. 1. Size, 0.379 × 1.512 ins. = 0.573 ± 2 sq. ins. O. H. Steel; standard bridge specifications. Strip from web of 15-in. channel. Ultimate strength = 62 000 lbs. Elastic limit = 38 000 lbs.

Load.	Time.	First scale reading.	Second scale reading.	Deflection, in centimeters.	Corrections:			Corrected deflection.	Deflection per 1 000 lbs.
					Temperature.	Rad. corrected to 1° 30′	Galvanometer.		
	m. s.								
4 070.....	1 42	40.97	49.65	8.68	0	+0.13	0	8.81	2.140
10 070.....	1 30	40.36	61.88	21.52	.....	.....	−0.04	21.48	2.133
11 350.....	1 25	40.12	64.38	24.26	.....	−0.17	−0.05	24.04	2.118
13 000.....	1 30	38.88	66.62	27.74	.....	.....	−0.08	27.66	2.128
15 075.....	1 31	38.47	70.60	32.13	.....	.....	−0.11	32.02	2.124
17 000.....	1 33	38.34	73.55	35.21	.....	+0.17	−0.15	35.23	2.072
18 000.....	1 39	38.17	75.25	37.08	.....	+0.46	−0.17	37.37	2.076
20 000.....	1 39	38.00	78.52	40.52	.....	+0.50	−0.19	40.83	2.041

Queen D'Arsonval galvanometer, and thermo-pile of 49 couples, used. Shellac used for attachment. Distance of scale = 246 cm.

C. A. P. Turner, Observer. A. Zeleny, Operator.

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EXPERIMENT 2.

Evening, August 8th, 1901.

Bar No. 2. Size, 0.378 × 1.488 ins. = 0.563 sq. in.

Load.	Time.	First scale reading.	Second scale reading.	Deflection, in centimeters.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad. corrected to 1·16".	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.
	m. s.									
5 960....	1 15	66.09	79.50	13.41	+0.05	−0.13	−0.09	−0.01	13.30	2.248
8 000....	1 15	66.61	84.66	18.05	+0.07	−0.18	−0.08	−0.03	17.89	2.236
9 920....	1 22	66.70	88.89	22.11	.....	0	+0.22	−0.04	22.29	2.237
11 040....	1 16	*66.70	91.38	24.68	.....	0	0.00	−0.05	24.58	2.227
12 100....	1 20	54.28	80.84	26.56	.....	0	+0.20	−0.06	26.70	2.207
13 180....	1 22	54.80	82.84	28.54	.....	0	+0.29	−0.08	28.75	2.219
16 050....	1 19	54.80	89.00	34.70	.....	.....	+0.19	−0.14	34.75	2.165
18 040....	1 12	54.00	92.90	38.90	.....	.....	−0.26	−0.21	38.85	2.153
19 500....	1 16	54.80	95.40	41.10	.....	.....	0.00	−0.23	40.87	2.096

Remarks.—At the end of the second reading the temperature suddenly fell about 3° Cent., due to a shower. Doors open between readings.

Distance of scale = 246 cm. Queen galvanometer and large thermo-pile.

Shellac contact.

C. A. P. Turner, Observer. A. Zeleny, Operator.

Bar cut from web of the 15-in. channel of O. H. steel; manufacturers' standard specifications. Ultimate strength = 60 000 lbs. Elastic limit = 39 000 lbs.

EXPERIMENT 3.

P. M., August 9th, 1901.

Bar No. 2 (*Continued*).—Stretched to 21 000 lbs., the evening of August 8th. Large thermo-pile and Queen galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad. corrected to 1·16".	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Rod.	Temperature: Room.
	m. s.											
8 085...	1 16	34.50	55.48	20.98	.....	.....	−0.07	−0.08	20.88	2.582	22.7°	23.0°
10 070...	1 17	34.68	60.31	25.63	.....	.....	−0.04	−0.06	25.53	2.535	22.7°	23.0°
11 000...	1 18	34.80	62.80	28.00	.....	.....	0.00	−0.08	27.92	2.535	22.7°	23.0°
12 100...	1 17	34.89	65.67	30.78	.....	.....	−0.04	−0.10	30.64	2.532	22.7°	23.0°
14 170...	1 20	34.91	70.08	36.12	.....	.....	+0.24	−0.16	36.20	2.556	22.7°	23.0°
16 000...	1 15	34.91	76.20	41.29	.....	.....	−0.14	−0.23	40.92	2.557	22.7°	23.0°
18 050...	1 18	35.25	79.82	44.57	.....	.....	0.00	−0.29	44.28	2.441	22.7°	23.0°
20 000...	1 27	34.70	80.69	45.99	.....	.....	+0.59	−0.33	46.25	2.312	22.7°	23.0°

Distance of scale = 246 cm.

C. A. P. Turner, Observer. A. Zeleny, Operator.

Remarks.—It will be noticed that these readings run quite regularly, the first a trifle high, perhaps, due in part to an observational error, and in part to the fact that the thermo-pile was affected by the lighting of the jets used for reading the scale. This effect would gradually disappear, the source of heat being constant.

The first reading was not in the center of the scale, and for that reason the galvanometer corrections are probably too small for loads of 14 000 lbs. and over.

It will be noted that the point where the curve changes has not been materially raised by the load applied.

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Experiment 4.

Evening, August 9th, 1901.

Bar No. 1 (*Continued*).—Strained to 24 000 lbs. Small thermo-pile and Knott galvanometer. Small Olsen machine used.

Load.	Time, seconds.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad. corrected to 30″.	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Rod.	Temperature: Room.
8 250...	30	62.41	53.40	9.01	−0.02	+0.01	.....	−0.07	8.93	1.082	22.5°	23.5°
5 940...	29	62.22	55.58	6.64	.....	.....	.....	−0.02	6.64	1.118	22.7°	24.0°
8 070...	26	62.22	55.39	8.83	.....	.....	.....	−0.06	8.77	1.087	.....	.....
6 900...	29	62.10	54.30	7.80	.....	.....	.....	−0.04	7.76	1.124	.....	.....
10 200...	29	62.34	50.87	11.47	.....	.....	.....	−0.15	11.32	1.110	.....	.....
10 000...	29	62.22	50.99	11.23	.....	.....	.....	−0.14	11.09	1.109	.....	.....
12 000...	29	62.20	48.80	13.40	.....	.....	.....	−0.22	13.18	1.098	.....	.....
12 000...	29	62.55	48.88	13.69	.....	.....	.....	−0.23	13.46	1.122	.....	.....
13 000...	32	62.39	47.68	14.71	.....	.....	.....	−0.27	14.44	1.111	22.7°	24.5°
15 000...	29	62.39	45.41	16.98	.....	.....	.....	−0.39	16.59	1.106	.....	.....
17 110...	30	62.50	43.02	19.48	+0.02	−0.02	.....	−0.51	18.97	1.108	23.0°	23.6°
19 080...	29½	62.31	45.53	16.78	+0.02	−0.02	.....	−0.37	16.41	0.862	.....	.....

Distance of scale = 249.2 cm. Resistance = 0 ohms.

C. A. P. Turner, Observer. A. Zeleny, Operator.

Remarks.—It may be noted that these readings are more irregular than can be accounted for otherwise than by the overstrain of the bar, and that the falling away of the curve from a straight line is approximately the same position as indicated in Experiment 1.

Experiment 5.

P. M., August 10th, 1901.

Bar No. 3.—Bar size = 2 x ½ in. Bar rough. Straight on one side and very slightly curved on the other. A thermo-pile on each side of the bar.

1. Large thermo-pile and Knott galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad. corrected to 1′ 8″.	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Temperature: Room.
	m. s.											
6 000	1 11	45.98	52.50	6.52	.....	.....	+0.04	−0.02	6.52	1.087	22.0°	22.0°
10 070	1 8	46.11	56.50	10.39	.....	.....	0.00	−0.11	10.28	1.021	....	....
10 080	1 7	46.30	55.92	10.62	.....	.....	−0.02	−0.12	10.48	1.045	....	....
14 050	1 8	45.59	61.51	14.92	.....	.....	0.00	−0.29	14.63	1.041	22.0°	22.1°
18 040	1 0	46.70	65.12	18.42	.....	.....	+0.27	−0.45	17.70	0.981	....	....
19 140	1 7	46.88	66.48	19.60	+0.01	.....	+0.03	−0.52	19.06	0.996	....	22.5°
20 200	0 55	46.80	67.50	20.70	+0.01	.....	+0.43	−0.58	19.70	0.975	....	22.5°
20 160	1 13	47.00	67.43	20.43	+0.01	.....	+0.19	−0.57	20.06	0.995	....	22.5°
22 300	1 19	47.21	71.42	23.21	+0.01	.....	+0.42	−0.75	22.89	1.031	....	22.5°
24 000	1 17	47.30	73.10	25.80	+0.01	.....	+0.40	−0.95	25.26	1.054	....	....
26 000	1 12	47.29	76.70	29.41	+0.01	.....	+0.20	−1.26	28.36	1.091	22.1°	22.5°
28 000	1 11	47.31	78.98	31.67	+0.02	−0.01	+0.19	−1.45	30.42	1.086	....	....
30 000	1 12	47.09	80 +	32.91	+0.02	−0.01	+0.26	−1.57	31.61	1.054	22.1°	22.5°

Distance of scale = 249.2 cm. Resistance = 0 ohms.

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2. Small thermo-pile and Queen galvanometer.

For headings of these columns, see previous page.

6 000	1 11	38.81	48.04	9.23	.....	.....	+0.06	−0.00	9.29	1.548	22.0°	22.0°
10 070	1 8	34.43	50.17	15.74	.....	.....	0.00	−0.02	15.72	1.561	....	....
10 040	1 7	35.21	50.70	15.49	.....	.....	−0.02	−0.02	15.45	1.538	....	....
14 050	1 8	30.05	51.68	21.63	.....	.....	0.00	−0.04	21.59	1.536	22.0°	22.1°
18 040	1 0	26.40	68.80	27.40	.....	.....	−0.36	−0.07	26.97	1.495	....	....
19 140	1 7	36.20	64.40	28.20	+0.01	.....	−0.04	−0.08	28.09	1.462	....	22.5°
20 200	0 55	36.72	65.10	29.38	0.00	.....	−0.61	−0.08	28.69	1.420	...	22.5°
20 160	1 13	37.38	65.20	27.82	+0.01	.....	+0.26	−0.08	28.83	1.430	....	22.5°
22 200	1 19	37.90	68.91	30.01	+0.02	.....	+0.57	−0.10	31.50	1.419	....	22.5°
24 000	1 17	38.10	69.00	30.90	+0.02	.....	+0.48	−0.10	31.30	1.304	.....	.....
26 000	1 12	38.29	74.10	35.81	+0.02	.....	+0.28	−0.16	35.95	1.383	22.1°	22.5°
28 000	1 11	38.10	77.30	39.20	+0.02	−0.1	+0.28	−0.21	39.23	1.401	....	....
30 000	1 12	37.90	77.70	39.40	+0.02	−0.1	+0.30	−0.21	39.50	1.317	22.1°	22.5°

Distance of scale = 246 cm. Resistance = 460 ohms.

C. A. P. Turner. Observer. A. Zeleny, Operator.

Remarks.—The bar, as noted, was a rough bar, and slightly kinked or bent on one side, and bending resulting therefrom may be noticed in the curve.

Experiment 6.

Evening, August 10th, 1901.

Bar No. 3 (*Continued*).—Bar stretched about 1½ hours to 24 000. Thermo-pile on each side of bar.

1. Small thermo-pile and Queen galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad. corrected to 1° 0'.	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Temperature: Room.
	m. s.											
8 050	1 4	38.20	50.40	12.20	.....	.....	+0.10	−0.01	12.29	1.526	22.0°	22.8°
12 150	1 2	38.71	56.39	17.68	.....	.....	+0.08	−0.02	17.74	1.461	....	....
10 285	1 0	38.90	54.32	15.42	.....	.....	0.00	−0.02	15.40	1.497	....	....
12 340	0 57	38.50	56.50	18.00	.....	.....	−0.12	−0.02	17.86	1.447	....	....
18 360	0 57	38.42	63.18	24.76	.....	.....	−0.16	−0.06	24.54	1.336	.....	.....
24 100	1 0	39·32	70.3	30.98	.....	−0.01	0.00	−0.10	30.87	1.281	22.1°	22.8°

Distance of scale = 249.2 cm.

2. Large thermo-pile and Knott galvanometer.

8 050	1 4	47.00	54.70	7.70	.....	.....	+0.06	−0.04	7.72	0.959	22.0°	22.8°
12 150	1 2	47.13	58.16	11.03	.....	.....	+0.05	−0.13	10.95	0.8987	....	....
10 285	1 0	47.21	56.89	9.68	.....	.....	0.00	−0.09	9.59	0.9766	....	....
12 340	0 57	47.42	58.81	11.39	.....	.....	−0.07	−0.15	11.17	0.9052	....	....
18 360	0 57	47.01	64.30	17.29	.....	.....	−0.11	−0.40	16.78	0.9140	....	....
24 100	1 0	47 21	68.10	10.89	.....	.....	0.00	−0.60	20.29	0.8454	23.1°	22.8°

Distance of scale = 246 cm.

C. A. P. Turner, Observer. A. Zeleny, Operator.

Remarks.—This bar had been strained to 35 000 lbs. per square inch and left under a load of 24 000 lbs. for 1½ hours.

The readings were quite irregular, due to this treatment.

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Experiment 7.

Evening, August 14th, 1901.

Bar No. 1 (*Continued*).—Bar annealed by heating 2 hours in oven. Thermo-pile on each side of bar. Gum-arabic contact, August 12th.

1. Small thermo-pile and Knott galvanometer.

Load.	Time, seconds.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad.	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Temperature: Room.	
8 180 (?)	30	43.56	57.12	13.56	....	....	Small.	—0.23	13.33	1.630	24.3°	25.0°	
8 210....	32	43.37	57.31	13.94	....	....	"	—0.25	13.69	1.675	.....	.....	
10 060....	30	43.54	59.59	16.05	....	....	"	—0.33	15.72	1.563	.....	.....	
10 135....	31	43.80	58.98	15.13	....	....	"	—0.29	14.84	1.464	.....	.....	
10 090....	27	43.85	54.78	10.85	....	....	"	—0.12	10.73	.....	24.3°	25.0°	Contact Defective.
14 000....	31	43.72	68.84	25.12	....	....	"	—0.90	24.22	.....	.....	.....	
12 115....	32	43.92	61.06	17.14	....	....	"	—0.40	16.74	.....	.....	.....	
16 050....	35	44.02	62.42	18.40	....	....	"	—0.46	17.94	.....	24.3°	25.0°	

Distance of scale = 249.2 cm.

2. Large thermo-pile and Queen galvanometer.

8 180 (?)	30	28.97	45.73	16.76	....	....	Small.	—0.02	16.74	2.046	24.3°	25.0°	.....
8 210....	32	29.50	46.02	16.70	....	....	"	—0.02	16.68	2.032	.....	.....	.....
10 060....	30	30.00	50.34	20.34	....	....	"	—0.03	20.31	2.019	.....	.....	.....
10 135....	31	30.27	50.46	20.19	....	....	"	—0.03	20.16	1.989	.....	.....	.....
10 090....	27	30.05	50.45	20.40	....	....	"	—0.03	20.37	2.022	24.3°	25.0°	.....
14 000....	31	30.33	58.29	27.96	....	....	"	—0.08	27.88	1.997	.....	.....	.....
12 115....	32	30.51	54.58	24.07	....	....	"	—0.05	24.02	1.987	.....	.....	.....
16 050....	35	30.68	62.02	31.34	....	....	"	—0.10	31.24	1.953	24.3°	25.0°	.....

Distance of scale = 246 cm.

Turner, Zeleny and Cates.

Remarks.—The above experiment was made using gum arabic junction, and, as noted above, the contact for the small thermo-pile was found defective before the end of the reading.

Experiment 8.

Evening, August 15th, 1901.

Bar No. 1 (*Repeated*).—Rubber cement contact. Cement put on about 4 hours before experiment.

1. Small thermo-pile and Knott galvanometer on one side of bar.

Load.	Time, seconds.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Corrections: Temperature.	Corrections: Rad.	Corrections: Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Temperature: Room.
8 180..	37	41.49	57.11	15.62	0.00	.....	Small.	—0.31	15.31	1.871	23.8°	24.6°
10 765..	35	42.20	62.61	20.41	+0.01	.....	"	—0.57	19.85	1.844	23.8°	25.0°
6 225..	37	43.80	54.24	11.92	0.00	.....	"	—0.17	11.75	1.888	.....	.....
12 240..	40	42.71	66.94	24.23	+0.01	.....	"	—0.84	23.40	1.912	.....	.....
13 440..	43	42.79	70.68	27.89	+0.02	.....	"	—1.12	26.79	1.993	23.8°	25.1°
16 560..	44½	42.78	77.20	34.42	+0.02	.....	"	—1.74	32.70	1.975	.....	.....
8 190..	39	42.69	60.28	17.59	+0.01	.....	"	—0.41	17.19	2.099	.....	.....
18 100..	38	42.61	80.49	37.88	+0.02	.....	"	—2.13	35.77	1.976	23.8°	25.1°
20 100..	41	42.80	85.50	43.20	+0.02	.....	"	—2.75	40.47	2.013	.....	.....

Distance of scale = 250 cm.

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2. Large thermo-pile and Queen galvanometer on other side of bar.

For headings of these columns, see previous page.

8 180..	37	47.65	62.55	15.90	0.00	.....	Small.	−0.02	15.88	1.941	23.8°..	24.6°
10 765..	35	47.37	68.48	21.11	+0.01	.....	"	−0.03	21.09	1.959	23.8°..	25.6°
6 225..	37	48.48	60.63	12.15	0.00	.....	"	−0.01	12.14	1.950	.....	.....
12 240..	40	48.30	72.10	23.80	+0.01	.....	"	−0.05	23.76	1.941	.....	.....
13 440..	43	48.32	74.95	26.63	+0.02	.....	"	−0.07	26.58	1.978	23.8°..	25.1°
16 560..	44½	48.50	79.97	31.47	−0.02	.....	"	−0.10	31.39	1.896	.....	.....
8 190..	39	48.73	65.02	16.29	+0.01	.....	"	−0.02	16.28	1.988	.....	.....
18 100..	38	48.73	84.10	35.37	+0.02	.....	"	−0.15	35.24	1.947	23.8°..	25.1°
20 100..	41	49.00	87.12	38.12	+0.02	.....	"	−0.19	37.95	1.850	.....	.....

Distance of scale = 246 cm. Turner, Zeleny and Skinner.

Remarks.—This experiment on Bar No. 1 when repeated was made with rubber cement junction. As the bar had not been overstrained since it was annealed, fairly regular results should have been expected.

There appears to have been some little bending in the bar, as will be observed by comparing the readings of the two sets. The first reading of the Knott galvanometer was higher than that at which the galvanometer correction was determined, namely, 80; hence these corrections are too small. Further, it will be noticed that the first readings of the Knott galvanometer were from 41.50 to 42.30, and a similar change in the first readings of the Queen galvanometer, thus showing considerable temperature change during the experiment.

Experiment 9.

Evening, August 17th, 1901.

Bar No. 3, stretched to 38 500 lbs. at 4.30 P. M., August 10th, and left under that stress until 5 P. M. of August 17th. Experiment began at 7.30 P. M., August 17th.

1. Small thermo-pile and Knott galvanometer. Larger Olsen testing machine used.

Small thermo-pile nearer lights, so that the readings are less accurate.
Resistance = 100 ohms.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Temperature.	Rad. corrected to 1° 10′	Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Room.
	m. s.											
4 120	1 0	28.20	30.59	2.39	.....	0.00	−0.04	0.00	2.35	0.5704	.....	.....
4 080	1 2	27.92	30.26	2.34	.....	0.00	−0.08	0.00	2.31	0.5662	25.1°	25.2°
6 000	1 5	27.69	31.24	3.55	.....	0.00	−0.03	0.00	3.52	0.5867	.....	.....
8 090	1 5	27.69	32.46	4.77	.....	0.00	−0.04	0.00	4.73	0.5847	25.05°	25.2°
10 080	0 48	25.82*	32.01	6.19	.....	0.00	−0.18	−0.01	6.00	0.5952	25.2°	25.0°
10 810	0 59	25.82	33.19	6.47	.....	0.00	−0.12	−0.01	6.34	0.5865	.....	.....
9 900	1 18	26.72	32.50	5.89	.....	0.00	+0.08	−0.01	5.96	0.6020	.....	.....
10 000	1 20	26.61	32.56	5.95	.....	0.00	+0.10	−0.01	6.04	0.6040	24.9°	25.0°
13 120	1 7	26.58	34.37	7.79	.....	+0.01	−0.05	−0.04	7.71	0.5877	.....	.....
16 320	1 8	26.45	36.22	9.77	.....	+0.01	−0.04	−0.06	9.68	0.5931	.....	.....
20 235	1 6	26.31	38.56	12.25	.....	+0.01	−0.08	−0.17	12.01	0.5934	.....	.....
24 170	1 6	26.07	40.52	14.45	−0.01	+0.01	−0.10	−0.27	14.08	0.5825	24.85°	24.8°
26 500	0 54	26.60	41.98	16.38	−0.01	+0.01	−0.38	−0.35	15.65	0.5902	.....	.....
30 308	1 10(?)	24.42	42.55	18.13	−0.01	+0.01	0.00	−0.44	17.69	0.5838	.....	.....
34 300	1 11	25.01	45.29	20.28	−0.01	+0.02	+0.08	−0.54	19.89	0.5767	24.85°	24.4°
38 350	1 5	24.42	47.30	22.94	−0.01	+0.02	−0.21	−0.74	22.00	0.5736	.....	.....

Distance of scale = 250 cm.

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2. Large thermo-pile and Queen galvanometer.

For headings of these columns, see previous page.

4 120	1 0	44.87	49.00	4.13	.....	0.00	−0.07	0.00	4.06	0.985	25.1°..	25.1°
4 080	1 2	44.73	48.83	4.10	.....	0.00	−0.06	0.00	4.04	0.980	.....	.....
6 000	1 5	44.68	50.82	6.14	.....	0.00	−0.05	0.00	6.09	1.015	25.05°	25.2°
8 090	1 5	44.54	52.78	8.24	.....	0.00	−0.07	0.00	8.17	1.010	25.2°..	25.0°
10 080	0 48	43.60	54.30	10.70	.....	0.00	−0.32	0.00	10.38	1.030	.....	.....
10 810	0 59	43.25	54.50	11.25	.....	+0.01	−0.20	0.00	11.06	1.020	.....	.....
9 900	1 18	43.36	53.38	10.02	.....	+0.01	+0.14	0.00	10.17	1.027	24.9°..	25.0°
10 000	1 20	43.30	53.38	10.08	.....	+0.01	+0.17	−0.10	10.16	1.016	.....	.....
13 120	1 7	43.37	56.85	13.48	.....	+0.02	−0.08	−0.01	13.41	1.022	.....	.....
16 320	1 8	43.19	59.90	16.71	.....	+0.02	−0.07	−0.02	16.64	1.020	.....	.....
20 235	1 6	43.17	63.83	20.66	−0.01	+0.02	−0.14	−0.03	20.49	1.013	.....	.....
24 170	1 6	43.07	67.40	24.33	−0.01	+0.02	−0.17	−0.05	24.12	0.998	24.85°.	24.8°
26 500	0 54	43.10	71.40	28.30	−0.01	+0.02	−0.67	−0.08	27.56	1.040	.....	.....
30 303	1 10(?)	42.20	72.55	30.35	−0.01	+0.03	0.00	−0.09	30.28	0.999	.....	.....
34 300	1 11	42.08	73.50	31.42	−0.01	+0.03	+0.05	−0.10	31.39	0.915	24.85°.	24.4°
38 350	1 5	42.10	79.90	37.80	−0.01	+0.04	−0.34	−0.19	37.30	0.972	.....	.....

Distance of scale = 24.6 cm. Resistance in box = 760 ohms. Bar No. 3 broken at 64 200 lbs.

C. A. P. Turner, A. Zeleny.

In this experiment it would seem probable that the effect of the treatment has been to reduce the cooling effect for all loads.

* A gas jet 12 ft. away caused the galvanometer needle to move 7 mm. in about 2 minutes, through three thicknesses of carpet. First reading changed from effect of this.

Experiment 10.

Evening, August 22d, 1901.

Bar No. 5. Surface planed. Area of bar = 0.445 × 2 ins. = 0.890 sq. in. Large thermo-pile and Queen galvanometer. Large Olsen machine used.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections:				Corrected deflection.	Deflection per 1 000 lbs.	Temperature:	
					Resistance.	Temperature.	Rad. corrected to 1' 0".	Galvanometer.			Bar.	Room.
	m. s.											
6 040	1 0	46.00	54.64	8.64	.....	.....	0.00	−0.00	8.64	1.430	Assumed constant.	25.05°
8 000	1 1	46.00	57.44	11.44	.....	.....	+0.02	−0.00	11.46	1.433		.....
12 000	1 3	46.36	63.36	17.20	.....	.....	+0.11	−0.02	17.29	1.441		*25.05°
15 520	1 11	46.67	68.39	21.72	.....	.....	+0.43	−0.03	22.12	1.425		.....
16 040	0 55	46.53	69.89	23.36	.....	.....	−0.23	−0.05	23.08	1.439		.....
15 825	0 57	46.58	69.59	23.01	.....	.....	−0.16	−0.05	22.80	1.441		.....
17 600	0 55	42.42	72.96	26.26	.....	0.00	−0.27	−0.06	25.98	1.473		.....
20 020	0 54	46.28	75.19	29.11	.....	0.00	−0.35	−0.08	28.68	1.433		25.0°
22 000	1 1	46.00	76.73	30.73	.....	+0.01	+0.05	−0.09	30.70	1.395		.....
24 000	1 0	45.90	79.40	33.50	.....	+0.01	0.00	−0.14	33.37	1.390		.....
25 000	1 0	45.68	81.21	35.53	.....	+0.01	0.00	−0.16	35.38	1.415		.....
24 200	0 59	45.55	79.78	34.23	.....	+0.01	−0.07	−0.14	34.03	1.406		.....
28 100	1 8	45.41	81.10	35.69	.....	+0.01	+0.23	−0.16	35.77	1.273		.....

* Application of load too slow at first weight?

Distance of scale = 250 cm. Resistance in box = 400 ohms.

C. A. P. Turner, A. Zeleny.

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Experiment 11.

Evening, August 26th, 1901.

Bar No. 5 (*Repeated*).—Bar strained to 28 000 lbs. on August 22d.

1. Small thermo-pile and Knott galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Temperature.	Rad. corrected to 1' 10".	Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Room.
	m. s.											
6 050..	1 26	31.60	36.21	4.61	.....	.....	+0.11	0.00	4.72	0.7801	23.4°	24.0°
9 865..	1 15	32.56	39.94	7.38	.....	.....	+0.07	−0.08	7.42	0.7523	.....	.....
10 065..	1 12	32.68	40.39	7.71	.....	.....	+0.03	−0.04	7.70	0.7650	23.4°	24.0°
6 073..	1 8	32.81	37.65	4.84	.....	.....	+0.02	0.00	4.82	0.7936	.....	.....
10 000..	1 13	32.80	40.29	7.49	.....	.....	+0.04	−0.08	7.50	0.7500	.....	.....
14 062..	1 0	32.35	43.00	10.65	.....	.....	+0.18	−0.12	10.85	0.7861	23.4°	24.0°

Distance of scale = 250 cm. Resistance in box = 130 ohms.

2. Large thermo-pile and Queen galvanometer.

6 050..	1 26	43.00	49.21	6.21	.....	.....	+0.15	0.00	6.36	1.026	23.4°	24.0°
9 865..	1 15	43.68	54.75	11.07	.....	.....	+0.10	−0.00	11.17	1.132	.....	.....
10 065..	1 12	44.22	56.35	12.13	.....	.....	+0.05	−0.01	12.19	1.184	23.4°	24.0°
6 073..	1 8	44.62	50.49	5.87	.....	.....	+0.02	0.00	5.85	0.963	.....	.....
10 000..	1 13	45.22	57.28	12.06	.....	.....	+0.07	−0.01	12.12	1.212	.....	.....
14 062..	1 0	45.70	62.17	16.47	.....	.....	+0.28	−0.12	16.17	1.150	23.4°	24.0°

Distance of scale = 246 cm Resistance in box = 460 ohms.

C. A. P. Turner, A. Zeleny.

Experiment 12.

Evening, August 27th, 1901.

Bar No. 7.—Area of bar = 0.440 × 1.68 ins. = 0.739 sq. in.

1. Small thermo-pile and Knott galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Temperature.	Rad. corrected to 1' 10".	Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Room.
	m. s.											
8 000..	1 15	32.71	40.90	8.29	.....	.....	+0.08	−0.05	8.32	1.040	24.48°	24.8°
6 010..	1 16	32.82	38.69	5.87	.....	.....	+0.06	−0.01	5.92	0.985	24.48°	24.8°
9 700..	1 12	32.44	42.50	10.06	.....	.....	+0.04	−0.10	9.92	1.033	.....	.....
14 230..	1 7	32.25	49.84	14 84	.....	.....	+0.09	−0.28	14.47	1.017	.....	.....
16 000..	1 6	32.09	48.50	16.41	.....	.....	+0.10	−0.36	15.95	0.997	24.50°	24.8°
17 000..	1 10	31.99	49.45	17.46	.....	.....	0.00	−0.41	17.05	1.003	.....	.....
20 000..	1 10	31.72	52.30	20.68	.....	.....	0.00	−0.58	20.10	1.005	.....	.....
23 000..	1 15	31.50	53.30	21.80	.....	.....	+0.20	−0.68	21.32	0.973	24.49°	24.7°

Distance of scale = 250 cm. Resistance in box = 130 ohms.

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2. Large thermo-pile and Queen galvanometer.

For headings of these columns, see previous page.

8 000..	1 15	46.32	57.67	11.35	.....	.....	+0.10	0.00	11.45	1.431	24.48°	24.48
6 010..	1 18	46.48	55.02	8.54	.....	.....	+0.12	0.00	8.66	1.441	24.48°	24.8°
9 700..	1 14	46.55	60.22	13.67	.....	.....	+0.10	−0.01	13.76	1.419	.....	.....
14 230..	1 7	46.25	66.52	20.27	.....	.....	−0.12	−0.03	20.12	1.414	.....	.....
16 000..	1 8	46.30	68.10	21.80	.....	.....	−0.09	−0.04	21.67	1.354	24.50°	24.8°
17 000..	1 10	46.00	69.76	23.76	.....	.....	0.00	−0.05	23.71	1.395	.....	.....
20 000..	1 10	45.90	73.02	27.12	.....	.....	0.00	−0.07	27.05	1.352	.....	.....
22 000..	1 15	45.40	74.28	28.88	.....	.....	+0.26	−0.68	29.06	1.321	24.49°	24.70°

Distance of scale = 246 cm. Resistance in box = 460 ohms.

C. A. P. Turner, A. Zeleny.

The temperature conditions during this experiment were not very satisfactory.

Description of Bar No 7: L2-4734, Boiler. Surfaces planed. Elastic limit = 41 000 lbs. Ultimate strength = 57 500 lbs. Elongation = 28½% in 8 ins. Analysis: Carbon, 0.21; Manganese, 0.88; Phosphorus, 0.083; Sulphur, 0.022.

This bar was strained up to the yield point in 1897, and had been allowed to rest until the date of the experiment, and the thermal limit of proportionality did not appear to have been raised.

Experiment 13.

Evening, August 29th, 1901.

Bar No. 8. Area of Bar = 0.433 × 1.62 ins. = 0.701 sq. in.

1. Small thermo-pile and Knott galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance.	Temperature.	Rad. corrected to 1° 18'.	Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Room.
	m. s.											
6 040	1 26	33.00	37.95	4.95	.....	.....	+0.06	0.00	5.01	0.829	23.5°	23.8°
8 100	1 17	33.13	39.87	6.74	.....	.....	−0.01	−0.02	6.71	0.827	.....	.....
10 060	1 21	32.79	41.14	8.35	.....	.....	+0.04	−0.05	8.34	0.829	.....	.....
11 780	1 16	32.60	42.51	9.91	.....	.....	−0.03	−0.10	9.78	0.8302	23.5°	23.7°
13 670	1 18	32.50	43.70	11.20	.....	.....	0.00	−0.14	11.06	0.8095	.....	.....
15 950	1 29	32.30	45.41	13.11	.....	.....	+0.21	−0.20	13.12	0.8226	.....	.....
18 000	1 19	32.19	46.65	14.46	.....	.....	+0.02	−0.27	14.21	0.7894	.....	.....
21 000	1 15	32.09	49.10	17.01	.....	.....	−0.09	−0.39	16.53	0.7871	23.5°	23.4°

Distance of scale = 250 cm. Resistance = 130 ohms.

2. Large thermo-pile and Queen galvanometer.

6 040	1 26	43.30	50.80	7.50	.....	.....	+0.09	0.00	7.59	1.256	23.5°	23.8°
8 100	1 17	43.50	53.82	10.32	.....	.....	−0.02	0.00	10.30	1.271	.....	.....
10 060	1 23	43.31	56.02	12.71	.....	.....	+0.11	−0.01	12.81	1.278	.....	.....
11 780	1 19	43.06	58.11	15.05	.....	.....	+0.02	−0.02	15.05	1.278	23.5°	23.7°
13 670	1 19	42.88	59.60	16.72	.....	.....	+0.02	−0.02	16.72	1.223	.....	.....
15 950	1 29	42.79	62.20	19.41	.....	.....	+0.32	−0.03	19.70	1.235	.....	.....
18 000	1 19	42.69	64.02	21.33	.....	.....	+0.03	−0.03	21.33	1.185	.....	.....
21 000	1 16	42.59	67.64	25.05	.....	.....	−0.08	−0.06	24.91	1.186	23.5°	23.4°

Distance of scale = 246 cm. Resistance = 670 ohms.

C. A. P. Turner, A. Zeleny.

Description: 4794 Fire-box L8. Elastic limit, 39 500 lbs. Ultimate strength, 59 000 lbs. Elongation, 28.7 per cent.

Analysis: Carbon	0.21	Phosphorus	0.035
Manganese	0.35	Sulphur	0.028

This bar also had been pulled to the yield point in 1897, and the load had been left applied for two days at that time, but the thermal limit of proportionality did not appear to have been raised after this period of rest.

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Experiment 14.

P. M., August 31st, 1901.

Bar No. 6. Planed. Area of Bar = 0.440 × 2 ins. = 0.880 sq. in.

1. Small thermo-pile and Knott galvanometer.

Load.	Time.	First reading.	Second reading.	Deflection.	Corrections: Resistance corrected to 20.5°.	Temperature. Rad. corrected to 1' 15"	Galvanometer.	Corrected deflection.	Deflection per 1 000 lbs.	Temperature: Bar.	Room.
	m. s.										
6 175	1 20	35.00	38.10	3.60	—0.01	.....+0.03	0.00	3.62	0.586	19.4°	19.5°
7 760	1 15	36.55	41.09	4.54	—0.01	..... 0.00	0.00	4.53	0.583	19.5°	19.9°
10 095	1 19	37.84	43.70	5.86	—0.01	.....+0.04	—0.01	5.90	0.584	19.5°	20.0°
14 040	1 19	39.97	48.24	8.28	—0.01	.....+0.06	—0.05	8.28	0.5897	19.6°	20.3°
16 050	1 15	40.68	49.93	9.25	.....	..... 0.00	—0.07	9.18	0.5719	19.8°	20.5°
17 800	1 18	40.65	50.95	10.30	.....	.....+0.06	—0.10	10.26	0.5764	19.8°	20.6°
19 800	1 14	40.27	51.94	11.67	.....	.....—0.02	—0.15	11.50	0.5808	19.8°	20.5°
21 950	1 17	40.41	53.41	12.87	.....	.....+0.05	—0.19	12.73	0.5800	19.8°	20.5°†
23 700	1 16	40.76	54.80	14.04	.....	.....+0.02	—0.25	13.81	0.5827	19.8°	20.5°
25 700	1 20	40.81	55.98	15.17	.....	.....+0.13	—0.29	15.01	0.5840	19.8°	20.5°
28 000	1 18	40.90	57.42	16.52	.....	.....+0.09	—0.33	16.28	0.5814	19.85°	20.5°
31 000	1 16	41.04	59.38	18.34	.....	.....+0.03	—0.45	17.92	0.5781	19.80°	20.5°
18 610	1 15	40.22	50.94	10.72	.....	..... 0.00	—0.10	10.62	0.5706	19.9°	20.5°
34 000	1 18	41.05	61.08	20.03	+0.01	—0.01+0.11	—0.54	19.60	0.5735	19.9°	20.6°

* Evening. † Chuck slipped.

Distance of scale = 250 cm. Resistance = 0 ohms.

2. Large thermo-pile and Queen galvanometer.

6 175	1 21	35.54	43.20	7.67	—0.01	.....	+0.07	0.00	7.73	1.252	19.4°	19.5°
7 760	1 14	43.15	53.01	9.86	—0.01	.....	—0.01	0.00	9.84	1.269	19.5°	19.9°
11 095	1 18	44.02	56.70	12.68	—0.01	.....	+0.09	—0.01	12.75	1.264	19.5°	20.0°
14 040	1 19	45.785	63.18	17.395	.....	.....	+0.12	—0.02	17.495	1.246	19.6°	20.3°
16 050	1 14	46.33	66.25	19.92	.....	.....	—0.03	—0.03	19.86	1.237	19.8°	20.5°
19 800	1 14	46.23	70.87	24.64	.....	.....	—0.04	—0.06	24.54	1.239	19.8°	20.6°
21 950	1 15	46.08	73.63	27.55	.....	.....	0.00	—0.07	27.48	1.252	19.8°	20.5°
23 700	1 13	46.12	75.20	29.08	.....	.....	—0.11	—0.08	28.89	1.219	19.8°	20.5°†
25 700	1 15	45.92	77.30	31.38	.....	.....	0.00	—0.10	31.28	1.217	19.8°	20.5°
28 000	1 15	45.93	80.14	34.21	.....	.....	0.00	—0.14	34.07	1.217	19.85°	20.5°
31 000	1 14	46.05	83.65	37.60	.....	.....	—0.06	—0.17	37.37	1.205	19.80°	20.5°
18 610	1 13	45.54	68.68	23.14	.....	.....	—0.08	—0.05	23.01	1.236	19.9°	20.5°
34 000	1 15	46.02	86.28	40.26	+0.01	—0.01	0.00	—0.22	40.04	1.177	19.9°	20.6°

Distance of scale = 246 cm. Resistance = 460 ohms.

Turner, Zeleny and Kennicott.

Remarks.—Large temperature changes. Rates of 0.02 to 0.08 had to be allowed for in most cases. Zero reading of the Knott galvanometer was high, and hence the corrections for the larger readings are too small in the upper set.

The foregoing curves are fair examples of what can be done by this method of thermal measurement under somewhat unfavorable conditions. Where a laboratory is especially fitted for thermal measurement, a more sensitive machine for weighing the loads is used, and

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where suitable means are provided for applying all loads uniformly and in more nearly the same time, a probable error not greater than 1 in 2 000 to 5 000 would perhaps be a reasonable limit of accuracy.

The method has a number of advantages over any other method of measuring stress with which the writer is familiar:

First. It is necessary only to have a space of about 1 sq. in. to apply the junction, and, indeed, this space might be reduced to ¼ sq. in. and secure almost equally good results; or the thermo-pile might be of special shape, long and narrow, for measuring fiber stress in columns under compression, if so desired. For the measurement of stress in railroad bridges the apparatus would appear to be very suitable. An error of ± 5 to 6% would be the probable accuracy when the effect of air currents, etc., is considered.

The angular deflection of the galvanometer might be determined in the laboratory for given intensities of stress applied to a standard bar, and, using the same apparatus in the field, with a similar junction, and with the same resistance in the circuit, the readings could be readily interpreted. Such investigation, however, would require time and patience, because, in order to secure reliable results, care must be taken that all connections are in good order, and, where the apparatus is not working nicely, some little ingenuity is required to locate the difficulty.

Where a piece of metal is subjected to compound stress (for instance, to tension in one direction and compression in another), the cooling effect of the tensile strain will be offset by the heating effect of the compressive stress, and the reading would evidently indicate the difference between the two.

A very interesting case of this kind of stress will be found in the examination of a plate-girder with a stiffened web. The usual text-book analysis of the internal stress in the web plate of such a girder is radically in error, and Figs. 2 and 3 present the writer's conclusions from a thermo-electric investigation of the internal stress in the small riveted girder shown in Fig. 4. The distribution of the stress through the web plate is in belts, and the position of these belts varies with the position of the stiffening angles. In the experimental girder, Fig 4, the load was applied by a jack-screw and an I-beam lever at one end and weighed off at the other.

Referring to the analysis of internal stress in beams, in Rankine's

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"Applied Mechanics," it is found that the case examined by him is that of a beam of constant section along its length. For such a beam he proves clearly that the principal stresses at the neutral axis are tensile and compressive, acting at 45° with this axis and at 90° with each other.

The special case, of a beam of a given type of cross-section, increasing or decreasing regularly in section along its length, is amenable to similar mathematical analysis, and the general distribution of the internal stress would vary but slightly from the first case considered, as stated in the "Applied Mechanics."

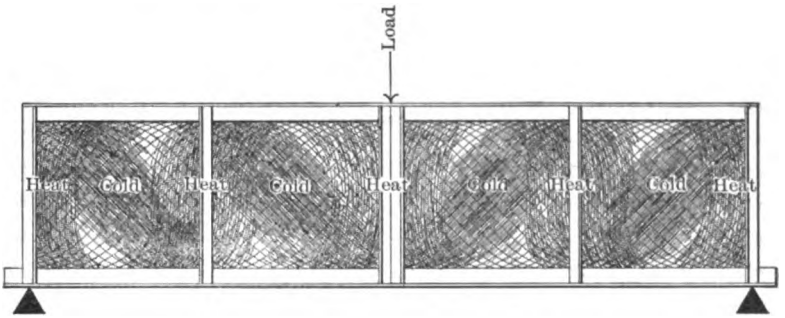

FIG. 2.

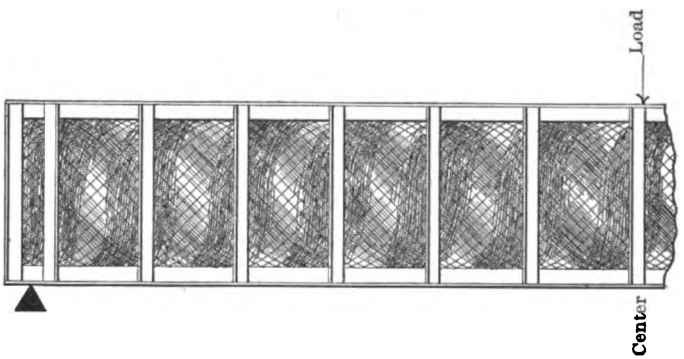

FIG. 3.

The general case, of a beam varying irregularly along its length, both as regards area and type of section, is quite a different problem, involving too many conditions to be handled in a general way. Even the special cases, when the variations are at regular intervals, with a constant section between, are by no means simple or easily analyzed.

Since built sections, when well riveted, act under stress approxi-

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mately as solid sections, the built girder with stiffeners should be considered as coming under beams of the latter class, and the writer would subdivide such girders into general classes as follows:

Class 1.—Those girders in which the stiffeners are spaced economically; that is, sufficiently close to prevent buckling of the web, and causing it to be capable of fully developing the strength of the flange.

Class 2.—Those girders in which the stiffeners are placed needlessly close together.

Class 1 includes those girders in which the designer has followed Cooper's specification, in which he requires stiffeners to be placed about the depth of the girder apart when the shearing stress on the web exceeds that allowed by the formulas given.

Referring to Fig. 2, it will be noted: (1) that the lines of maximum compressive stress in the web are concentrated, as it were, in the dark area near the stiffeners, and that they have changed in direction from 45°, being deflected toward that portion of the plate best able to resist compression by virtue of the lateral support of the stiffener angles; (2) that the path over which the maximum stresses act is but slightly longer than it would be were the tensile and compressive stresses acting at 45° with each other; and (3) it is evident that the intensity of the stresses in the dark area in excess of those in the other portion of the plate is a direct function of the size of the panel, and an inverse function of the thickness of the plate.

Were the stiffeners applied vertically, and so close together that the web is practically stiffened all over, it would undoubtedly be in a condition of internal stress similar to that of the plate of a flitched beam, which the common theory covers exactly. For cases between this extreme condition of excessive stiffening and that of stiffeners spaced economically, as investigated, intermediate conditions of internal stress would naturally be expected. A solution will be offered for one only of these, as giving a fair insight to the treatment proposed for other cases.

Referring now to observations 1 and 2, concerning the distribution of stress shown in Fig. 2, these deductions may be summarized in the statement that a load seeks the shortest course consistent with the strongest members, or parts.

Referring now to Fig. 3, as in Fig. 2, the belts or areas of maximum compression would be expected near the stiffener, and slightly inclined

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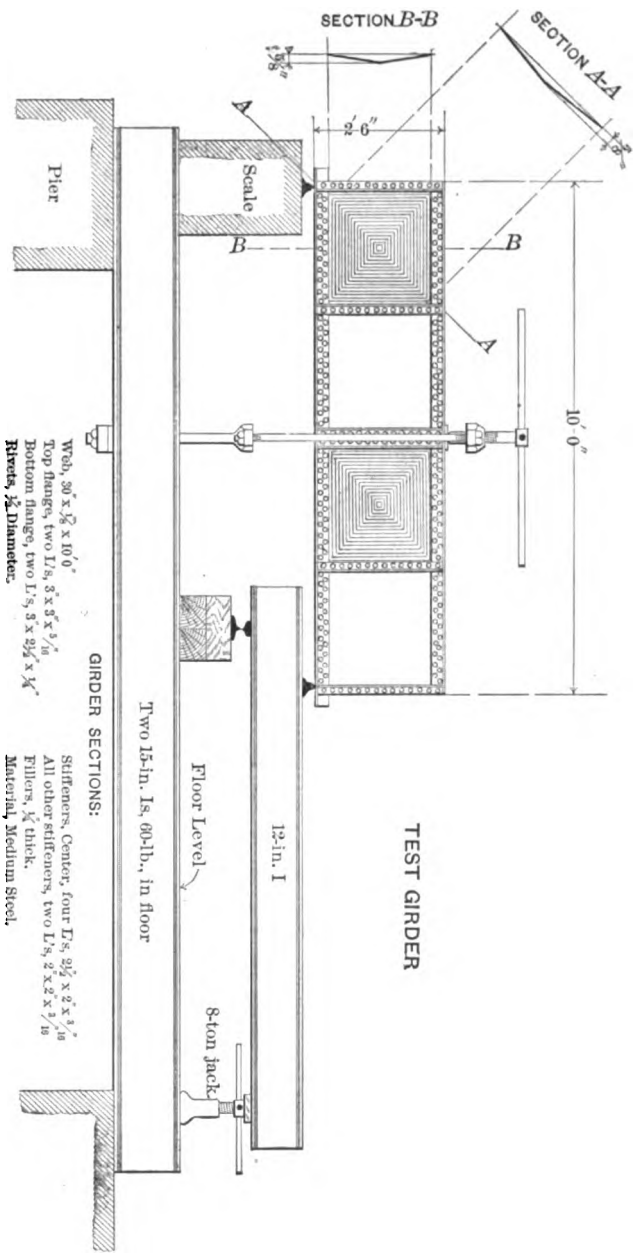

Fig. 4.

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to it, while the belts of maximum tension, which act in connection with them, in order to fulfill the condition of following that path which would permit the least vertical deflection of the structure would naturally run to the alternate panel point, this being the shortest path, consistent with the condition named, that will allow the maximum compressive stresses to follow those portions of the plate which by the lateral support of the stiffeners is best able to resist them.

The point of most interest in this case, with a distribution of stress as outlined, would be that the stiffener would be strained much less than in the first case, Fig. 2, since the stiffener does not follow the center line of either belt of maximum stress, but, lying between them, the component of the distortion of the compressive belt along the length of the stiffener is offset in a measure by that of the tensile belt of maximum stress across it at the center.

The action of the stiffener in affecting the distribution of stress in the web may be regarded as two-fold:

First. In rendering the plate in its vicinity better able to resist compressive stress through the lateral support it affords, thus causing an uneven distribution of stress in the web, as shown in Fig. 2.

Second. Under favorable conditions, it may relieve the web of a considerable portion of the compressive web stress.

For the former action, few rivets are required, but to act efficiently in the latter capacity, where the stiffeners are economically spaced, they should be well riveted.

The effect of this distribution of the web stress in belts should, in the writer's judgment, be taken into consideration in proportioning the flanges, particularly where the girders are short and deep, and, furthermore, in deciding on the spacing of the rivets.

Measurements by Method B.—When a bar is loaded, the elastic distortion is opposed and limited by the internal molecular forces; in other words, it is said that the internal and external forces are in equilibrium. This at once conveys the conception of a different general or average distribution of internal stress between the molecules and groups of molecules along different axes, dependent on the direction of the external constraining force, and the question naturally arises whether the material will present different properties along different axes by which we may measure nearly or approximately this induced condition of internal strain.

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Thomson has answered this question, partially, at least, by showing that the thermo-electric quality of iron, under pressure, deviates from that of the unstrained metal toward bismuth for currents in the direction of the strain, and toward antimony for currents perpendicular to this direction. While other metals examined by him—zinc, brass, copper and steel—showed uniformly the reverse effect to that of iron when similarly treated.*

If the position of a bar in the thermo-electric series is changed by strain, then this change may be detected by clamping to the strained bar a piece of similar unstrained metal and forming a circuit between the two bars, within which a galvanometer may be placed to measure the thermo-electric current generated by heating the junction of the two pieces of metal. Then, if the change is proportional to the load or the strain on the bar, or bears a known relation to it, this provides the necessary means of measuring the stress.

The writer's first attempt to measure stress in this way was made as follows: A 2 x ½-in. bar was placed in the testing machine, and another similar bar was clamped to it at right angles or horizontally, using a wood clamp. To the upper end of the bar in the machine was attached a steel wire, by wrapping a few turns about the bar and clamping with an iron clamp. Another steel wire was attached in a similar manner to the end of the horizontal bar, and both wires were then connected with the Queen D'Arsonval galvanometer.

The junction of the two bars was then heated with a bag of hot water, the bag being wrapped around the junction. This caused no deflection whatever, there being no load on the bar. Upon applying a load of 15 000 lbs. gradually, there was noted a gradual deflection of 1½ mm., which disappeared as the load was removed.

As this deflection was much too small to be satisfactory, an endeavor was made to get a larger deflection by using a Thomson astatic galvanometer, instead of the D'Arsonval, but the instrument was so badly disturbed by the ground currents induced by the street-railway lines as to be entirely useless. Thus, the only thing that remained to do was to either put in a coil of less resistance in the D'Arsonval galvanometer or apply more heat to the junction.

The former would have been the desirable thing to do, as it would

* *Proceedings*, Royal Society, London, 1854–55. *Philosophical Transactions*, Royal Society, London, 1855–56.

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have secured accurate work, and would have cost only about $15. The time required to get the coil, however, about three weeks, caused the the writer to use the latter alternative.

Wrapping the junction with asbestos paper, it was heated up to about 180° with a gas jet, and allowed to cool to about 150° or thereabouts. Heating the junction with the bar under no load caused no deflection, but as soon as the load was applied a deflection was secured roughly in proportion to the load, for the loads up to about 18 000 or 22 000 lbs. per square inch, beyond which the increase in deflection was at a far smaller ratio than the increase in the load.

The results secured are given in part in the following tests. In considering them it should be borne in mind that the junction was gradually cooling by radiation, and the extent of the rate may be determined roughly by the slight change in deflection for the smaller loads applied at the latter part of the tests. These tests are very easy to make, requiring less than 5% of the time necessary for direct thermal measurement. The essential element necessary for satisfactory work is a constant source of heat for the junction, and securing a similar piece of metal to that which is being investigated, to form the junction.

Set 1.

Load.	Deflection:			Rate per 1 000 lbs.
	Load applied.	Load removed.	Average.	
11 100	0.63	0.58	0.605	0.0575
21 000	0.70	0.70	0.70	0.0334
22 100	0.60	0.82	0.71	0.0322
10 150	0.47	0.70	0.583	0.0574
81 000	0.90	1.00	0.95	0.0306
10 750	0.57	0.58	0.575	0.0534

C. A. P. Turner, Observer.

Set 2.

12 000	0.50	0.43	0.465	0.0388
28 200	0.66	0.68	0.67	0.0291
30 000	0.84	0.82	0.83	0.0276
14 200	0.46	0.61	0.53	0.0373
88 000	0.84	0.90	0.87	0.0229

A. Zeleny, Observer.

Set 3.

16 000	0.51	0.51	0.51	0.051
12 000	0.60	0.60	0.60	0.050
15 000	0.78	0.76	0.75	0.050
19 000	0.1	1.01	1.005	0.053
23 360	1.05	1.05	1.05	0.045

C. A. P. Turner, Observer.

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The readings of Set 3 are unquestionably the most reliable, as the bars had had time to become of nearly a uniform temperature, which may perhaps account for the higher point at which the rate of deflection decreases.

These few crude experiments should not, however, be taken as a basis for definite conclusions, further than that the method appears to be practicable, though requiring considerable careful work to place it on an equally satisfactory basis with that by direct thermal measurement.

This method, which has been classed as "*B*" in the early part of the paper, like Method "*A*," appears to throw an interesting light on the change in molecular structure of the metal, under such stresses as exceed well-defined limits. As the temporary strain changes the thermo-electric quality of the material, so also does a permanent strain, but the residual effect of the permanent strain or set is the reverse of that which subsists as long as the constraining force is kept applied.

Thomson concludes that the peculiar thermo-electric qualities thus induced are those of a crystal. Thus, he finds that iron bars hardened by longitudinal compression have the reverse thermo-electric property to that discovered by Magnus in wires hardened by drawing, and that iron under lateral compression manifests the same thermo-electric property that he had discovered in an iron wire under a longitudinal stretching force.

Having noted the characteristics of the change in the molecular structure of the material, caused by severe stress, it may naturally be inquired, at what intensity of stress this change commences. It has been noted that it becomes very apparent whenever the load has been sufficient to give the metal a permanent set, and, as such yielding is invariably accompanied by the evolution of heat, the inference is natural that the commencement of this change in structure begins when the thermal curve ceases to be a straight line.

When the repetitions of severe stresses are of one kind only, it may be conceived that there is a gradual change in the structure of the bar of a somewhat homogeneous character, with the ultimate result that the life of the piece would be long under Wohler treatment. But, on the other hand, where the range of stress is from tension to compression, since the residual effects of the two kinds of severe stress are

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of the reverse order, a correspondingly shorter life, due to the lack of homogeneity in the change in structure by the two kinds of severe stress, should be expected.

What is the nature of this residual effect or change of the molecular structure? It has been noted that the temporary effect of mechanical strain of lower intensity is to change the thermo-electric property of the metal, and, apparently with reason, it has been attributed to

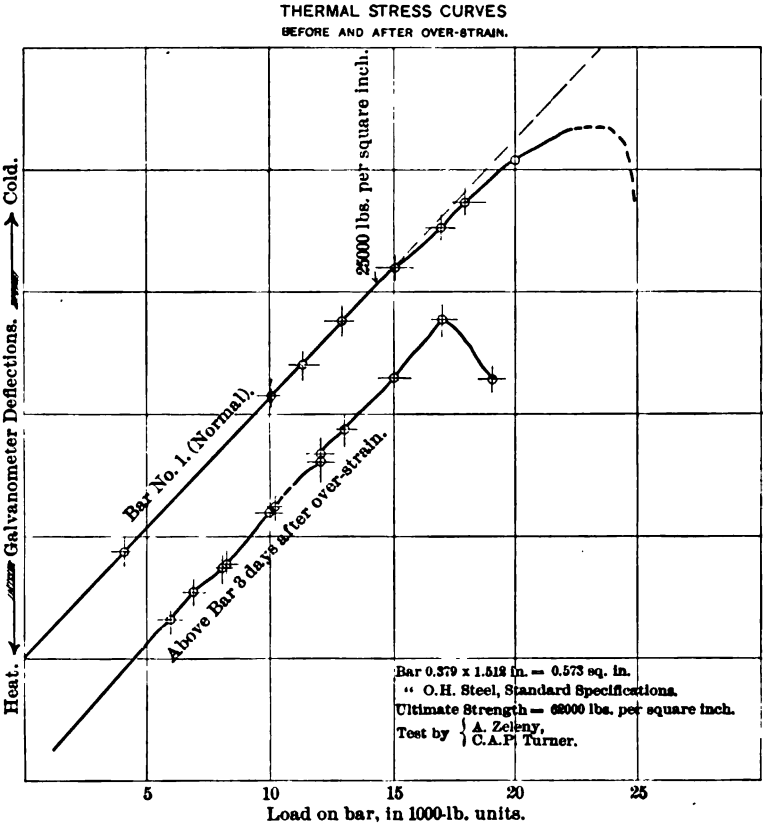

FIG. 5.

an unequal distribution of the inter-molecular forces opposing the external forces. Arguing, then, that similar effects are produced by similar causes, these residual effects may perhaps be attributed to a condition of permanent internal strain along certain lines or axes, the natural result of which would enable it more readily to resist stress of the kind which produced this condition, while, at the same time ren-

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dering it weaker in other directions, less elastic or resilient and more likely to be fractured by shock or sudden loading.

In concluding this paper, a few general remarks regarding the elastic condition of the metal investigated may not be amiss, and a careful consideration of the curves presented in Fig. 5 will give a fair idea of the results to be expected under ordinarily careful work, depending upon whether the specimen is in a truly normal or unstrained condition, or whether the internal structure has been disturbed by recent overstrain. If the latter condition is that with which we have to deal, regularity of results cannot be expected, and the writer's work gives some ground for supposing that there is a slight difference in the amount of the cooling effect due to the stretching, for nearly all loads, it being less in cases where the material has been overstrained.

In the tests presented, less attention has been paid to the condition of the bars than in determining the most practical type of junction to use. Further, the writer regards the behavior of a galvanometer under a load continuously applied as a somewhat more satisfactory indication of its elastic properties than the measurements which may be taken consecutively and presented as has been done in the tests given herein.

The writer takes pleasure in acknowledging the assistance received in the experimental work from Mr. Anthony Zeleny, Instructor in Physics at the University of Minnesota, and from Messrs. F. G. Skinner, A. T. Fay, H. A. Kennicott and F. E. Cates.

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DISCUSSION.

Mr. Turner. C. A. P. TURNER, M. Am. Soc. C. E. (by letter).—As there has been no formal discussion on this paper, the writer will add some data and try to cover questions which have been asked by various members.

The resistances thrown in the circuit were not recorded in some of the first experiments, as there was some trouble with internal contact of the piles. The following data will aid in comparing the experiments:

Resistance of Queen D'Arsonval, at 22° Cent., 237.6 ohms.
" " Knott " " 22° " 224 "
" " Large Thermopile* 20 "
" " Small " * 5.7 "

Temperature coefficient of copper wire = 0.00388 per degree Centigrade.

The following examples of the calculation of the resistance correction are given to show the method of arriving at the results, the corrections in each case being given only to the nearest tenth of a millimeter.

Page 165, Part 1, first line.

$20.5° - 19.5° = 1.0°$; $0.004 \times 1.0° = 0.004$, correction fraction.

$3.60 \times 0.004 = 0.0144 \backsimeq 0.01$;† the sign must be minus, as the deflection was too large when the temperature was lower than 20.5 degrees.

Page 165, Part 2, first line.

$20.5° - 19.5° = 1.0°$

$0.004 \times 1.0° = 0.004$, correction fraction for copper coil;

$$0.004 \times \frac{237}{460 + 237} = 0.0013$$ correction fraction for circuit;

$7.67 \times 0.0013 = 0.00997 \backsimeq 0.01$.† The sign is minus for the same reason as in the previous case.

The "dead beat" type of galvanometer, of course, was used in the work, that is, one in which the needle or coil is sufficiently damped to come to rest at the proper deflection without vibrating back and forth. This is accomplished in the D'Arsonval type by means of currents induced in the metal frame upon which the coil is wound.

In the Thomson type the suspended system is enclosed in a chamber, and the oscillations are damped by the pumping of air from one side of the chamber to the other by the discs to which the astatic system is attached.

It has been noted that the current has been generated in the pile by the change in temperature of the bar affecting, by radiation, the

* Varied considerably in first tests, due to defective contact.
† Corrections given to nearest tenth of a millimeter.

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temperature of one end of the pile. As it requires an increment of time for the temperature change to affect the pile to its full amount, it also requires a considerable time for the two ends of the pile to drift back to the same temperature after taking a reading, as a current will continue in the circuit as long as there is any difference in temperature between the two ends of the pile. Mr. Turner.

Where very accurate work is attempted, measurements can only be made at about 20-minute intervals. However, where an error of 1 or 2% is immaterial, we may note the rate and direction of the drift of the reading and allow for it in the corrected deflection. In the experiments given, the operators waited until the drift was practically zero.

In this lies the practical advantage of Method *B* over Method *A*, as with Method *B* the readings change almost as quickly as the stress on the bar, and successive readings can be taken without involving the errors noted. On the other hand, the difference in thermo-electric intensity between the strained and the unstrained metal with Method *B* is so slight that we have to deal with extremely weak currents, which it is possible to measure satisfactorily only with the very sensitive galvanometer constructed specially for thermal work. In this form the resistance of the coil is made comparatively small, usually about 10 ohms. This is accomplished by winding the coil with several wires in parallel, instead of with one continuous wire.

It has been remarked by one member that the deflections per 1 000-lb. load run very uniformly in each test, but vary greatly in the different tests.

As this discrepancy is more apparent than real, it may be well to reduce one or more of the readings of Experiments 10, 11, 12 and 14 to the same basis of deflections per 1 000 lbs. per square inch for the same resistance in the circuit and the same temperature of the bar, and then compare them.

Take the second part, and the first line, of each test, and we have the results shown in the following table:

Experiment No.	Deflection per 1 000 lbs. on bar.	Section of bar, in square inches.	Deflection per square inch.	Temperature.	Resistance in box, in ohms.	Deflection per 1 000 lbs. per square inch reduced to equivalent of 460 ohms resistance and 25.05°Cent.
10.....	1.430	0.89	1.27	25.05°	400	1.165
12.....	1.431	0.739	1.156	24.48°	460	1.157
14.....	1.253	0.88	1.10	19.4°	460	1.12 (1st line).
14.....	1.269	0.88	1.115	19.5°	460	1.135 (2d line).
11.....	1.026	0.89	0.915	23.4°	460	0.916 (Same bar as 10 after over-strain).

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Mr. Turner. In reducing Experiments 10 and 12 to the same basis, the total resistance is estimated at 664 ohms in Experiment 10, and 724 ohms in Experiment 12.

It is noted that, with the different bars used in these two experiments, the unit deflections reduced to the same basis are within seven-tenths of 1% of each other, a difference which may be due to several causes:

1st. Errors in work, or determining resistance;

2d. Difference in thickness of rubber insulation; or

3d. Difference in the composition of the bar, or the treatment to which it has been subjected.

The examples from Experiment 14 do not run as close as these, but in this there is a difference of 1½% between the first and second readings, and the way they run in the test indicates some bending, or disturbing condition, such as air currents, etc.

Experiments 10 and 11 are on the same bar, and show a difference of 25% in the cooling rate per unit of stretching force, which appears to be most readily accounted for on the ground of the immediately previous over-strain leaving the structure of the metal in a disturbed condition, with the result that any distortion, however small, would be accompanied by some internal friction and heat, offsetting a part of the cooling effect of the stretching.

Not a few of our engineering textbooks assume that the molecular structure of the metal undergoes no change by repeated over-strain, because it shows little difference under ordinary tests, and no apparent change in structure when etched with acids and examined under a powerful magnifying glass. Bearing in mind, however, the approximate determinations of molecular magnitudes by the different methods of Clausius and Kelvin, conclusions based upon such crude and coarse methods of investigation appear to be of little weight, and entirely unwarranted in the light of general experience. That the metal, as a mass, if given a permanent set, presents different qualities along different axes, analogous to a crystal, is apparently well established, but whether there is such change in structure as is caused by improper heat treatment may be open to question.

In advancing the suggestion that this change in structure was something of the nature of an induced condition of internal stress, we have some proof, perhaps, in the stated experiences of some mechanical engineers with cold-rolled shafting, namely, that they have had a straight piece of this shafting, when key-seated the full length, become somewhat curved—concave along the key-seat—further, that this curvation developed gradually with the piece lying at rest.

That the writer's criticism of the dynamic theory should have brought out no discussion is somewhat surprising, in view of the fact that a no less generally used specification than that of Mr. Cooper is based directly upon it.

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Mr. Turner.

In presenting his dynamic theory, however, Mr. Fidler should be credited with an able discussion of the incongruities of fatigue formulas, as applied to lower intensities of stress, leading to such extreme results, if we allow reasonably for the dynamic effect of the live load, which, by hypothesis, they are not designed to cover, that we are naturally led to discredit them entirely.

In conclusion, the writer desires to state specifically that he regards the thermal method of investigation as perfectly practical for actual measurements of stress in railroad bridges, but that the errors in doing so would be much greater than in laboratory work, and would probably be in the neighborhood of plus or minus 4 or 5% with ordinarily careful work.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 921.

THE SUPPORTING POWER OF PILES.*

By Ernest P. Goodrich, Jun. Am. Soc. C. E.

With Discussion by Messrs. E. Sherman Gould, Horace J. Howe, Joseph P. Carlin and Ernest P. Goodrich.

The use of piles in the construction of foundations dates from primitive times. The villages of the prehistoric tribes dwelling on the lakes in Switzerland were almost entirely on piles. The very existence of Venice depends on piles, and modern engineering makes use of millions of piles each year.

Formerly, piles were simply sticks of wood stuck into the mud, but modern engineering makes use of iron and steel piles, and of other formations such as the so-called sand and concrete piles. These are used for various purposes. They often stand with one end upon a firm footing and the other supporting the mass to be carried. In this case the pile acts simply as a column with more or less lateral support.

Sometimes, piles are used simply to make more compact the soil into which they are driven, through the natural compression suffered by the earth which is displaced by the penetration of the pile. The bearing power of the soil is thus increased, about in proportion to the

* Presented at the meeting of February 5th, 1902.

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ratio of the sectional area of the piles driven, over a given area, to the latter. Usually, however, the piles act directly against the forces to be sustained, and are held in place simply by the resistance of the soil into which they penetrate. This resistance, under increase of the external forces, may prove too small to equilibrate the latter, and movement of the pile thereby results. Piles of the latter class are used sometimes in close clusters, sometimes in close rows, but usually singly. They may be square, hexagonal, round, or of other sections; may be uniform in size, or tapering; pointed, or blunt, etc. They may be put in place by means of water jets, or by continuous pressure, or by the rapid blows of a steam or gunpowder driver, or the slower, more measured blows of the ordinary gravity hammer.

This discussion will be confined solely to the use of the common, tapering, round, wooden pile, driven by the ordinary gravity hammer, which is raised by a rope on a steam drum, falls by its own weight upon the head of the pile and thereby forces it to its destined place, where it is held simply by the contact of the surrounding soil. Further, only the extreme sustaining power of the quiescent loads of piles immediately after being driven as above described, and in such positions as to allow each pile to act by itself, will be discussed. In these times, the determination of this question is very important, and, though considerable has been done along this line, the subject still remains in an undeveloped state as compared with most similar modern engineering questions, and of necessity, perhaps, must ever remain so.

An instructive experiment, throwing some light on the nature of the support given to a pile by the earth into which it is driven, is to take a box with a glass front, fill it with lightly compacted sand, and push down between the glass and the earth half-round sticks, similar to piles in shape. This experiment discloses the fact that a compact cone of earth is formed under the foot of a blunt stick, and remains there, being pushed forward through the ground as the stick descends. This cone acts exactly like the sharp end of a pointed stick or pile. It will always form under any load which soil is required to carry, and, consequently, the bearing power of the latter is one of the elements helping in the support of a loaded pile.

In most soils this bearing power is known to increase with increase in depth below the surface, and it would, therefore, be expected that

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the bearing power of a pile would increase as it is driven deeper and deeper into the ground. Therefore, if a curve were to be constructed, showing the supporting power of a pile for different penetrations, it should, theoretically, start from a point, the co-ordinates of which are penetration = zero, supporting power equals supporting power of soil, and be a straight line, inclined at some angle with the axes of reference, depending upon the rate of increase in the bearing power of the soil.

The penetration of the stick into the sand discloses, around the pointed end, flow lines along which the earth moves as it is pushed aside and compressed by the penetration of the stick. The extent of the region throughout which movement occurs in this way depends upon the compressibility of the soil at that point. Theoretically, this compressibility should decrease with increase of depth beneath the surface, but the actual variation is so slight as in nowise to affect the supporting power of piles. The displaced soil in contact with the pile is pressed against the latter by its own elasticity and by the influence of the stresses in the surrounding earth. Theoretically, the resistance to motion of the pile from this source should increase directly as the depth below the surface, so that the curve of supporting power already described must be compounded with a second straight line, starting from the origin, and slanting at a certain angle with the axes of reference, depending upon the amount of friction observed. The actual amount of this frictional resistance varies greatly with many circumstances.

The fact that piles enter loose ground and mud with equal penetrations for equal blows, however far driven, would tend to show that the frictional resistance was so small as not to be observable, and that the main resistance was in this case the supporting power of the ground beneath the pile. On the other hand, where the pile has been driven between stones or logs into hard or compact soil, the frictional resistance offered must often be far greater than all other means of support. The above-mentioned small frictional resistance, however, may be greatly increased after lapse of time by the compacting of the earth around the pile from the driving of others near it, or through the natural settlement back against the pile of the soil forced away while it was being driven.

When a pile is supported entirely by the frictional resistance, the

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actual region supporting the load is some deep ground level at which the frictional resistance holding the pile has been transferred through the earth in the shape of a conoid of pressure, the base of which gives a total bearing value equal to the load and a unit bearing value which the earth at that lower level will support. Each kind and degree of compactness of earth will give a different angle for the slope of the conoidal surface. When the frictional resistance is relatively small, more of the pile must be in the ground, or the pile will settle through the immediately surrounding earth. Under excessive load, if the bearing power of the earth is small and the frictional resistance rather high, the pile will carry down the earth surrounding and in contact with it.

As the exact nature of the soil is seldom known, any criterion based upon the same would be valueless in determining the supporting power of a pile. The engineer can be guided by any information he may have on this point only in the spacing of the piles one from another. When the piles are held by the real bearing power of the soil they may be driven solidly if thought best, and in any case they will act simply as columns. When supported by frictional resistance, they must be driven so far apart, or to such a depth, that the increased area of bearing developed by the conoid of pressure having the required altitude of frictional resistance meets a level which will afford the required support before intersecting the conoid of a neighboring pile.

No criterion for the supporting power can be determined with absolute certainty, because one cannot say that after whatever test may be made, a pile will act under the next identically similar test in exactly the same way. The probabilities are very great, however, that if similar conditions are observed as far as possible, a pile under two similar tests will act in nearly the same manner, and with perhaps a little less probability that two neighboring piles will act alike when subjected to similar tests.

If a pile is found to begin to sink under a given quiescent load applied several days after it has been put in place, it will probably begin to sink a second time at nearly the same load if similarly applied. The probabilities are nearly as great that a neighboring pile, showing the same phenomena while being driven, will carry the same load. This probability must, therefore, be the basis for any practical determination of the question. Of course, the most satisfactory way

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is to actually test as many as possible of the piles under question with quiescent loads.

It is a great pity that more numerous determinations of this character have not been made in the past, or, if made, that the records of the same have not become available to engineers. Almost all the information so far published is to be found in "Piles and Pile Driving," edited by the late A. M. Wellington, M. Am. Soc. C. E., who quotes extensively from a compilation* by John C. Trautwine, Jr., Assoc. Am. Soc. C. E., and to which attention is herewith especially directed. Another valuable paper† is entitled "Some Instances of Piles and Pile-driving, New and Old," by Horace J. Howe, M. Am. Soc. C. E.

As no criterion for supporting power can be obtained from the soil, the phenomena observable during the driving of a pile must be examined.

A study of the various phenomena involved in the blow of a falling body striking a pile and forcing it into the ground is intricate. The relation of the phenomena to the ultimate supporting power of the pile is at best quite uncertain. It is the relation of a case of impact to one of quiescent pressure.

A body free to fall under the influence of the attraction of the earth does so with an accelerated motion, thereby increasing in kinetic energy. If this body strikes another, both are compressed to amounts depending upon their elasticity, and the second is set in motion when the amount of compression has reached such a point that its force overcomes whatever force of hindrance may exist. The latter force tends to bring the second body to rest, or both bodies, provided their relative elasticities are such that they have not separated. This force may be constant or variable, and probably at the last instant of its action its value would equal in amount the ultimate resistance offered by this second body to being moved by a slowly and steadily increasing force.

The following mathematical treatment is believed to cover as thoroughly as possible the various phases of the problem.

Let V = velocity of hammer at pile head (actual);

g = acceleration due to gravity;

h = height of fall of hammer.

Then $V = (2gh)^{\frac{1}{2}}$, theoretically.

* *Transactions*, Am. Soc. C. E., Vol. xxvii, pp. 148-160.

† *Journal of the Association of Engineering Societies*, Vol. xx, p. 257.

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Let $V = (b\,h)^m + x$, practically, as found by experiment;

w = velocity of hammer and pile at instant when both move together.

Then $V - w$ = velocity lost by hammer.

Let M_h = mass of hammer.

Then $M_h g$ = weight of hammer.

Let W_h = weight of hammer.

Then $\frac{W_h}{g} = M_h$.

Let M_p = mass of pile;

W_p = weight of pile;

M_g = mass of earth moved in connection with pile;

W_g = weight of earth moved in connection with pile.

Then $\frac{W_p + W_g}{g} = M_p + M_g$;

$\frac{W_h}{g} V$ = momentum of hammer at pile head;

$\frac{W_h}{g} (V - w)$ = momentum lost by hammer;

$\frac{W_p + W_g}{g} w$ = momentum gained by pile and earth moved by it.

But $\frac{W_h}{g} (V - w) = \frac{W_p + W_g}{g} w$;

$$w = \frac{W_h}{W_h + W_p + W_g} V.$$

Let $\frac{W_h}{W_h + W_p + W_g} = R_w$, be ratio of weight of hammer to that of pile and hammer combined.

Then $w = R_w [(b\,h)^m + x]$.

Let u = varying velocity of pile after instant pile and hammer move together.

Let $u = (K w)^r + y$.

Then $u_o = \left\{ K R_w [(b\,h)^m + x] \right\}^r + y$ is initial velocity.

Let t = time occupied in stopping motion of pile;

p = penetration of pile;

$p = C t - z$, be law of variation of penetration with time.

Then $\frac{dp}{dt} = u$.

$u = n C t^{n-1}$.

$\frac{du}{dt} = n (n - 1) C t^{n-2}$ is acceleration at any instant.

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Let f = force bringing pile to rest.

Then $f = (M_h + M_p + M_g)\frac{d\,u}{d\,t}$.

M_h will equal zero if hammer and pile separate immediately after instant both move as one.

$$f = (M_h + M_p + M_g)\, n\,(n-1)\, C t^{n-2},$$

$$\frac{f}{u} = \frac{(M_h + M_p + M_g)\, n\,(n-1)\, C t^{n-2}}{n\, C t^{n-1}}.$$

$$\frac{f}{u} = (M_h + M_p + M_g)\frac{n-1}{t}.$$

$$t = (M_h + M_p + M_g)\frac{(n-1)\, u}{f}.$$

$$\frac{u}{p+z} = \frac{n\, C t^{n-1}}{C t^n}.$$

$$\frac{u}{p+z} = \frac{n}{t}.$$

$$t = \frac{n\,(p+z)}{u}.$$

$$(M_h + M_p + M_g)\frac{(n-1)\, u}{f} = \frac{n\,(p+z)}{u}.$$

$$f = (M_h + M_p + M_g)\frac{(n-1)}{n}\,\frac{u^2}{p+z}.$$

Let f_o = initial force.

Then $f_o = (M_h + M_p + M_g)\frac{n-1}{n}\,\frac{u_o^2}{p+z}$.

$$f_o = (M_h + M_p + M_g)\frac{n-1}{n} \times \frac{\{K R\,[(b\,h)^m + x]\}^r + y}{p+z}$$

Let z = penetration lost through crushing of head, heating of head, compressing pile and hammer;

v = work done in crushing head, heating head, etc.;

q = quantity of work done in compressing pile and hammer.

When the hammer strikes the pile, the pressure between the pile and the hammer, which tends to again separate the same, will increase from zero up to the value of f_o, at which instant the pile as a whole will begin to move.

Let $a f_o$ be average of force up to instant pile and hammer move together;

e' = coefficient of elasticity of material of hammer;

L' = average length of hammer.

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Then $\frac{L'}{e'}$ = total compression per unit of compressive force for each unit area.

Let s' = average sectional area of hammer;

p' = average force throughout length of hammer tending to compress same at each instant.

Then $\frac{p'}{s'}$ = force per unit area of hammer;

$\frac{p'\,L'}{s'\,e'}$ = total compression suffered by hammer;

$a f_o \frac{p'\,L'}{s'\,e'}$ = work done in compressing hammer.

Let $p' = c' f_o$;

Then $a\,c' f_o^2 \frac{L'}{s'\,e'}$ = work done in compressing hammer.

Let e = coefficient of elasticity of material of pile;

L = average length of pile;

s = average sectional area of pile;

c = fraction denoting portion of f equaling average pressure throughout length of pile tending to compress same.

Then $a\,c\,f_o^2 \frac{L}{s\,e}$ = energy consumed in compressing pile;

$$q = a f_o^2 \left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right);$$

$$z = \frac{q + v}{f_o};$$

$$z = a f_o \left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right) + \frac{v}{f_o};$$

$$f_o = (M_h + M_p + M_g)\frac{n-l}{n} \times \frac{[\{K R_w [(b\,h)^m + x]\}^r + y]^2}{p + a f_o \left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right) + \frac{r}{f_o}}$$

Let F = final force;

$$F = j f_o;$$

$$F = \frac{-jp}{2a\left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right)} + \frac{j}{2a\left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right)} \times$$

$$\sqrt{p^2 + 4a\left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right)\left(\frac{W_h + W_p + W_g}{g}\right)\frac{n-1}{n}} \times$$

$$\sqrt{\left\{K R_w [(b\,h)^m + x]\right\}^r + y^2 - 4av\left(\frac{c\,L}{s\,e} + \frac{c'\,L'}{s'\,e'}\right)}.$$

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This expression is exceedingly long and unwieldy, but, before discussing how it may be shortened, a glance at its relation to other proposed formulas will be of interest. Rudolph Hering, M. Am. Soc. C. E., in a monograph entitled "Bearing Piles,"* has collected and collated fourteen different formulas. Those there given, together with some others, are here reproduced for reference, together with their derivation from the writer's formula by the substitution of certain values for the unknowns.

Trautwine: $60\ W_h\ ^3\sqrt{h}$ (if p is very small).

$$\frac{5 W\ ^3\sqrt{h}}{p+1}.$$

McAlpine: $80\ \{W_h + (0.228\sqrt{h} - 1)\ 2\ 240\}$.

Rankine: $$2\left\{-\frac{S\,e\,p}{L} + \sqrt{\frac{S\,e\,W_h\,h}{L} + \frac{s^2\,e^2\,p^2}{L^2}}\right\}.$$

Weisbach:

$$\frac{s'\,e'\,s\,e}{c\,L\,s'\,e' + c'\,L'\,s\,e}\left(\sqrt{p^2 + 2\,\frac{c\,L\,s'\,e' + c'\,L'\,s\,e}{s'\,e'\,s\,e}\,W_h\,h} - p\right).$$

Redtenbacher: $$-\frac{e\,p}{L} + \sqrt{\frac{2\,e\,W_h^{\,2}\,h}{s\,L\,(W_h + W_p)} + \left(\frac{e\,p}{L}\right)^2}.$$

Brix and Becker: $$\frac{W_h^{\,2}\,h\,W_p}{p\,(W_h + W_p)^2}.$$

Weisbach: $$\frac{W_h^{\,2}\,h}{p\,(W_h + W_p)} + (W_h + W_p).$$

Weisbach: }
Mason: } $$\frac{W_h^{\,2}\,h}{p\,(W_h + W_p)}.$$

Weisbach: }
Sanders: }
Molesworth: } $$\frac{W_h\,h}{p}.$$ (Sanders gives $3\,p$.)

Nystrom: $$\frac{W_h^{\,3}\,h}{p\,(W_h + W_p)^2}.$$

Baker:

$$\sqrt{W_h\,h\,\frac{12\,s'\,e'\,s\,e}{3\,L'\,s\,e + 4\,L\,s'\,e'} + \frac{36\,p^2\,s'^2\,e'^2\,s'\,e'}{(3\,L'\,s\,e + 4\,L\,s'\,e)^2}} - \frac{6\,p\,s'\,e'\,s\,e}{3\,L'\,s\,e + 4\,L\,s'\,e'}.$$

* Eng. News Pub. Co., New York, 1878.

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Wellington: (*Engineering News* formula.) $\dfrac{W_h h}{p+1}$

Crowell: $\dfrac{W_h h}{p+0.1}$ + etc.

Hurtzig: $\sqrt{500\ W_h h + (250\ p)^2} - 250\ p.$

A glance at these formulas reveals the following facts:

Most authors neglect friction in the fall of the hammer, so that in the writer's formula $b = 2g$, $m = \frac{1}{2}$, $x = 0$.

Most authors consider that the pile and the hammer do not separate; then $u = w$, and $r = 1$, $K = 1$; $y = 0$.

All neglect energy lost by heating, etc.; then $v = 0$.

They consider, without so stating, that initial and final forces are equal; then $j = 1$.

They also consider, without so stating, that the penetration varies as the square of the time; then $n = 2$.

They also consider $W_g = 0$.

With these assumptions:

$$F = \frac{-p}{2a\left(\frac{cl}{se} + \frac{c'l'}{s'e'}\right)} + \frac{1}{2a\left(\frac{cl}{se} + \frac{c'L'}{s'e'}\right)}\sqrt{p^2 + 4a\left(\frac{cL}{se} + \frac{c'L'}{s'e'}\right)W_h h R_w}.$$

Weisbach assumes, besides the foregoing, that the pressure is uniform throughout the pile and the hammer; then $c' = c = 1$. He also assumes that the pressure increases uniformly from zero up to f, then $a = \frac{1}{2}$. His formula, therefore, is

$$F = \frac{s'e'se}{cLs'e' + c'L'se}\left(\sqrt{p^2 + 2\frac{cLs'e' + c'L'se}{s'e' + se}W_h h} - p\right).$$

Rankine neglects compression in the hammer, that is, $e' = \infty$. He also assumes that the pressure throughout the pile varies uniformly; that is, $c = \frac{1}{2}$. He also assumes $a = \frac{1}{2}$.

His formula, therefore, is

$$F = \sqrt{\frac{4se\,W_h h}{L} + \frac{4s^2e^2p^2}{L^2}} - \frac{2sep}{L}.$$

Baker assumes that the pressure in the hammer varies uniformly; that is, $c' = \frac{1}{2}$; and he takes $a = \frac{1}{2}$. He also assumes c to be $\frac{2}{3}$,

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stating that the average pressure is probably below the center of the pile.*

His formula then becomes,

$$F = \sqrt{W_h h \frac{12 s' e' s e}{3 L' s e + 4 L s' e'} + \frac{36 p^2 s'^2 e'^2 s' e'}{(3 L' s e + 4 L s' e')^2}} - \frac{6 p s' e' s e}{3 L' s e + 4 L s' e'}.$$

This he reduces to

$F = 100 \left(\sqrt{W_h h + (50 p)^2} - 50 p \right)$ by assuming 5 000 for the quantity $\frac{6 s' e' s e}{3 L' s e + 4 L s' e'}$.

Hurtzig's formula is the same as the latter, except for the numerical quantities.

Redtenbacher assumes the first mentioned values for all constants except K, and also that $e' = \infty$, $\frac{2 a c}{s} = 1$. He makes $K = \frac{\sqrt{2}}{\sqrt{s}}$, and finds that

$$F = \frac{-p e}{L} + \sqrt{\left(\frac{p e}{L}\right)^2 + \frac{2 e W_h^2 h}{s L (W_h + W_p)}}.$$

Many authors neglect the compression of the pile and hammer entirely; then $e = e' = \infty$. Letting, also, $W_g = 0$, $v = 0$, $j = 1$, $n = 2$, $a = \frac{1}{2}$, as before, and as most writers do, we have

$$F = \frac{W_h + W_p}{2 p g} \left[\left\{ K R_w \left| (b h)^m + x \right| \right\}^r + y \right]^2.$$

Nystrom's work seems to take $x = 0$, $y = 0$, $K = 1$, $r = \frac{3}{2}$, $m = \frac{1}{3}$, $b = 2 g$; that is, in fact, that the friction is such as to make the velocity of fall vary as the cube root of the height and u vary as $W^{\frac{1}{3}}$. His formula is, then,

$$F = \frac{W_h^3 h}{(W_h + W_p)^2 p}.$$

Again, if $u = \sqrt{R_w w}$, and $V = 2 g \left(h + \frac{p}{R^2_w} \right)$, then $y = 0$, $r = \frac{1}{2}$, $K = R_w$, $m = 1$, $b = 2 g$, $x = \frac{2 p g}{R_w^2}$, $F = \frac{W_h^2 h}{p (W_h + W_p)} + W_h + W_p$, which is one of Weisbach's forms. He reaches it through an entirely different course of reasoning, however.

* "Treatise on Masonry Construction," page 237.

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Brix and Becker assume $x = 0$, and $K = \frac{R_{ic}\, W_p}{W_h + W_p}$, instead of as in the last case, and find $F = \frac{W_h^2\, W_p\, h}{(W_h + W_p)^2 p}$.

Mason's work assumes no friction; that is, $b = 2g$, $m = \frac{1}{2}$, also, $r = 1$, $K = 1$, $x = 0$, $y = 0$. This gives

$$F = \frac{W_h^2\, h}{(W_h + W_p)\, p},$$

which is also one of the forms given by Weisbach.

Many authors neglect the weight of the pile entirely. If, in the last case, $W_p = 0$, $F = \frac{W_h\, h}{p}$, which is nearly Sander's formula, and Weisbach's final form.

Another group of formulas, having a constant besides p in the denominator, may be obtained as follows:

As before, let $v = 0$, $j = 1$, $n = 2$, $a = \frac{1}{2}$, $x = 0$, $y = 0$, $K = 1$, $r = 1$, $b = 2g$, $W_g = 0$. And, also, if the pile and hammer are such that $\frac{c\, L}{s\, e} + \frac{c'\, L'}{s'\, e'} = \frac{2\, N}{f_o}$, or $N = \frac{g}{f_o}$, then the value of F is

$$\frac{W_h + W_p}{2g} \left\{ \frac{W_h}{W_h + W_p} (2gh)^m \right\}^2 \frac{1}{p + N}$$

Of course, N may be assumed as a quantity of any degree of complexity

If, now, $m = \frac{1}{2}$ and $W_p = 0$, $F = \frac{W_h\, h}{p + N}$, which is practically Crowell's form.

If the velocity varies as the sixth root of the fall, $m = \frac{1}{6}$, and if the weight of the pile be neglected, $W_p = 0$, and if $N = 1$,

$F = \frac{\text{constant } W_h \sqrt[3]{h}}{p + 1}$, which is nearly Trautwine's formula.

If friction be neglected, $m = \frac{1}{2}$, then if $W_p = 0$, and $N = 1$, $F = \frac{W_h\, h}{p + 1}$, which is really Wellington's or the *Engineering News* formula, except for the factor of safety.

Having thus obtained a general formula, which, in one or another of its modified forms, is the one used by different authors in determining the supporting power of a pile, it remains to be seen what effects the various assumptions made by them actually have on the results obtained.

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The wide variation observable proves conclusively that some of the assumptions made are seriously in error. Actual experimental determination of the various quantities entering the theoretical formula is the only way to find the true value in a given case. The writer's formula, being entirely theoretical, involves many quantities, the values of which have not hitherto been ascertained. The first efforts made by the writer along this line were to find the law of variation of force, the time interval occupied by the blow of the hammer, and the movement of the pile. In other words, the values of n and t were to be ascertained.

In the report of Brevet Lieut.-Col. James L. Mason, Corps of Engineers, U. S. A., concerning the foundation of Fort Montgomery, the following statements appear:

"The calculation (of the supporting power of a pile) is very easy, by the supposition that the retarding force is constant during the very short period that it takes to destroy the motion of the pile. This time was not measured. To measure it might be a difficult mechanical problem. Now, the variation of the force may be according to a simple or complex law. Its variations may be steadily in one direction, that is, constantly (not uniformly) to increase or constantly to diminish the intensity that the force had at the beginning of the motion, or they may be the reverse, tending during one portion of the time to increase, and at another to diminish the value of the force at the commencement of the motion."

In connection with the construction of the sub-foundation of the Sherman Statue, situated just south of the Treasury Building in Washington, D. C., a machine was contrived which should show besides the exact amount of the vertical motion of the pile, the time occupied by this motion, the velocity of the hammer as it struck the pile, and of the pile at each instant of its movement, and also the amount of compression suffered by the head of the pile from the blow of the hammer.

This apparatus (Fig. 1) consisted of a frame which could be held near the head of the pile. It was at first held in position by a vertical leg resting on the ground and two braces at right angles fastened to the frame and resting on the ground. Later, it was held by suitable arms resting against the guides in which the hammer of the pile-driver moved.

The second method was more readily managed, but the apparatus

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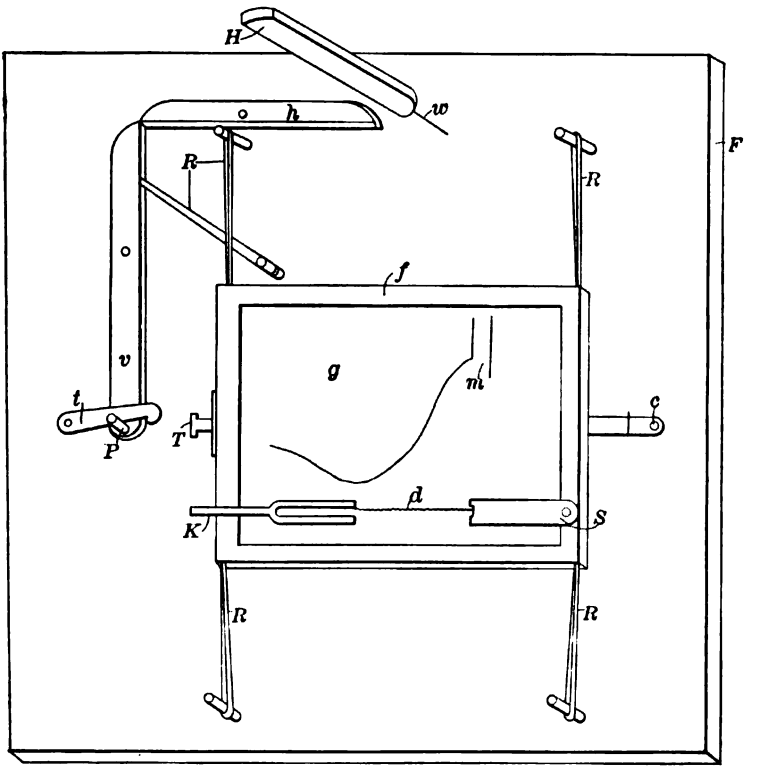

F. Frame of Apparatus.
f. Frame of Smoked glass.
g. Smoked glass.
c. Catch.
t. Trigger.
T. T-shaped dog.
v. Vertical lever.
h. Horizontal lever.
R.R. Rubber Bands.
K. Tuning fork.
S. Strip to release fork.
P. Peg to raise trigger.
H. Stick in hammer.
w. Wire to mark on glass.
m. Marks of wire.
d. Sinusoid.

FIG. 1.

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suffered somewhat from the jar of the frame-work of the derrick. Moving in front of this frame was a piece of smoked glass, about 6 by 8 ins. on a side, rigidly held in a light wood frame. This latter was supported by four rubber bands fastened to the four corners of the frame surrounding the glass, and stretched vertically above and below those corners to four pegs set in the frame of the apparatus. These rubber bands could be so adjusted on the several pegs that the glass might move horizontally, parallel with the frame, and when drawn aside and released would vibrate from side to side horizontally, moving in a vertical plane parallel with the frame of the apparatus.

At one side was a catch to stop and hold the glass at the end of a single vibration. On the opposite end of the frame holding the glass was a T-shaped metal dog serving to engage the trigger which, when tripped, released the frame, allowing it to vibrate after it had been drawn to that side and set. The trigger was simply a notched piece of metal pivoted on a level with the dog. This trigger was released by a peg in the end of a vertical lever, so pivoted that, as the peg moved around the fulcrum, it would raise the trigger. The upper end of the lever was held against the end of a second horizontal lever by a strong rubber band.

This second lever was pivoted near its center, so that, when its free end was lowered, its outer end released the vertical lever which in turn released the trigger, thus allowing the glass to make a single half vibration, and be caught by the catch on the other side of the frame. The horizontal lever was struck, as the hammer fell, by a stick projecting from a hole in the latter.

Projecting from the end of the stick was a wire which made a mark on the smoked glass as the hammer fell and as the glass moved. A tuning-fork was placed horizontally in front of the glass, so that a wire fastened to one of the prongs would scratch a path on the glass as the latter moved. When the glass had been set, and was held by the trigger, the prongs of the tuning-fork were pressed together and held in that position by a strip of metal with a square notch in its end, into which the prongs fitted. The other end of this strip was fastened to the frame holding the glass, so that when the latter began its half vibration the strip was pulled off, allowing the fork to vibrate and thus trace a sinusoid upon the smoked glass. The horizontal motion of the glass, theoretically, would be accelerated first positively and

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then negatively, but the sinusoid traced by the fork showed an almost absolutely uniform motion. From a test, the fork was afterward found to vibrate 500 times a second.

In each observation taken with the machine, the hammer was allowed to remain upon the head of the pile after a blow, and the apparatus was then so adjusted that the wire in the end of the stick attached to the hammer was about 2 ins. below the top of the smoked glass. The hammer was then raised to the desired height, the trigger set, and the hammer allowed to fall. The slant of the line upon the glass, from its top down to the point where the hammer struck the pile, would measure the velocity with which the hammer struck.

In the observations here described, this could not be relied upon to give true results because of the large amount (about 2 ins.) of lateral motion possible by the hammer in the guides. At the instant of striking the pile, the slant of the line, theoretically, should change, making a greater angle with the vertical, thus showing the reduced velocity of the system composed of the pile and hammer. As to the remainder of the motion, nothing could be predicted, as no one knew the law of force according to which the pile moved. However, theory predicted that there would be a slight rise in the curve after becoming horizontal, due to the reaction against the compression of the head of the pile, due to the impact of the hammer.

A study of the forms of the curves obtained is very instructive, and brings to light many facts as to the action of piles under the effect of the blow of a pile-driver hammer. What may be called a typical diagram is shown in Fig. 3. In it, the line, $A\ B$, is that made by the hammer in the last 2 ins. of its fall; and its slant, tan. $A\ B'\ D'$, should give the velocity of the hammer when it struck the pile. This point of striking is at B. The curve, $B\ C\ D$, shows the velocity of the pile at each instant of its descent. The vertical distance, $B\ D'$, shows the penetration of the pile, $B\ E'$, and the compression, $E'\ D'$, suffered by the head of the pile under that blow.

The time occupied is measured by the horizontal distance, $D'\ D$, measured in terms of the sinusoid above. At the point, D, the pile ceased to descend, and the vertical distance, $E\ D''$, shows the compression which the head of the pile underwent, as measured by the reaction of the pile head. This reaction occupied an interval of time measured by the horizontal distance, $D\ D''$. At the point, E, the

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hammer left the pile in the rebound, and the line, EF, was drawn in the irregular motion of the rebounding hammer. According to this diagram, the velocity of the hammer was about 57 ft. a second, corresponding to a fall of 59 ft., whereas, in reality, the fall was 20 ft. The pile sunk 2.56 ins., and occupied about $\frac{47}{1000}$, or 0.047, second in doing so. The compression of the head was about 0.05 in. and the reaction took place in about $\frac{30}{5000}$, or 0.006, second.

Fig. 4 is a diagram taken on a pile with a very much broomed head. The amount of compression suffered by the pile is here very large, and, consequently, the point at which the hammer struck the pile is nearly lost because the motion was first taken up by the head; and the pile, as a whole, partook of the motion only gradually. Figs. 3 and 4 were taken from piles when their points were penetrating a stratum of soft, dry, loamy clay; and the diagrams are very smooth and regular. From these two figures, it would appear that the law of variation of velocity is such that the penetration, measured from the deepest point, varies as the square of the time measured from the final instant. That is,

$$p \propto t^2, \quad \text{or} \quad p = c\,t^2.$$

If c be chosen as 0.3, on the scale of the diagram,

$$p = 0.3\,t^2,$$

and the dotted curve of Fig. 3, which shows the locus of this equation, coincides almost exactly with the diagram. Necessarily, the value of this constant will change for different kinds and degrees of compactness, density, etc., of the soil being penetrated, and with the size and shape of the pile being driven.

Figs. 5, 6 and 7 are diagrams taken when the point of the pile was penetrating more heterogeneous material, clay mixed with gravel. Fig. 7 is an interesting study of the action of a pile penetrating such a stratum. For the first two or three thousandths of a second, the pile did not move perceptibly, but almost immediately increased, with a rapid acceleration, until it had sunk about 0.85 in., when it encountered an increased resistance which slackened its speed to some extent. It gained slightly in velocity as it penetrated that obstacle, but rapidly fell off after a few thousandths more, and came to rest in obedience to the law above enunciated, after sinking 2.20 ins. in all, and occupying about 0.034 second in the process.

Occasionally, a pile would appear to sink with a uniform velocity,

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as shown in Fig. 8. This velocity sometimes appeared to be exactly that of the falling hammer, and the diagram would take the form shown in Fig. 9. It would seem as if the pile were penetrating a soft stratum and suddenly struck some obstacle, for it was several times observed that the diagram of the next blow was similar to Fig. 7.

Fig. 10 is a diagram of a type occasionally encountered. It seems to be simply a modification of Figs. 8 and 9. From the position of the apparatus, it was evident that the hammer struck the pile at a point near the star in the figure. The pile seemed to respond instantly to the hammer, but to fall off to some extent in velocity, down to the point when it encountered the hard stratum. The foregoing diagrams were all taken from among the last three or four blows which each pile received, and in every case it seemed to be sinking about the same distance under each of any number of similar blows, however deep it was driven.

At a later date, a heavy cylinder was mounted upon pivots and made to rotate rapidly by the act of pulling a string wound around one end, as a top is spun (Fig. 11). This rapidly rotating cylinder was held so that a wire, firmly attached to the pile to be driven, would make a mark on the smoked paper on the surface of the cylinder. Some of the results obtained are shown in Figs. 12, 13 and 14. No relation seems apparent between the curves obtained with the wire on the hammer and with it on the pile.

Having thus obtained numerous graphical records of the pile's movement, a very careful study was required to determine the exact law of variation. The writer was fortunate enough to have access to a Coradi's graphical integrating machine. With the help of this instrument it was possible to draw the curves which are the first, second, etc., differentials of the curves first obtained. These are shown in Figs. 15 and 16. While the original curves are necessarily irregular, and doubtless the exact law of variation in no two cases is exactly identical, and in no case is uniform throughout the whole extent of motion; still, it would seem, from a study of the curves, as if the variation of the penetration was as the square of the time in the average case, and that it was best to make that important primary assumption in the building of a formula. The curves also show that in the majority of cases the final intensity of force was the same as the initial intensity. This, in the writer's general formula, makes $n = 2$ and $j = 1$.

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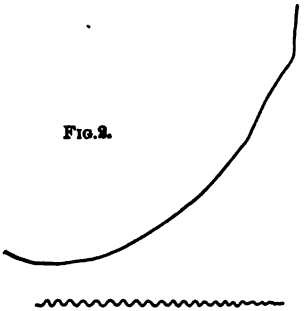

FIG. 2.

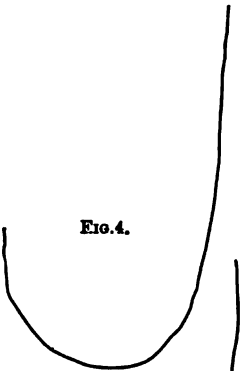

FIG. 4.

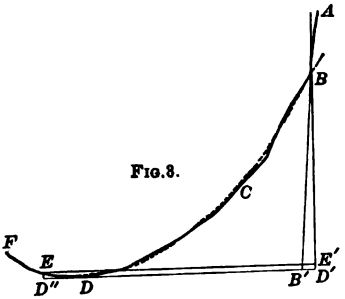

FIG. 3.

FIG. 5.

FIG. 10.

FIG. 6.

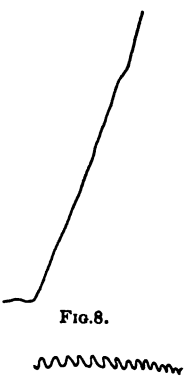

FIG. 8.

FIG. 7.

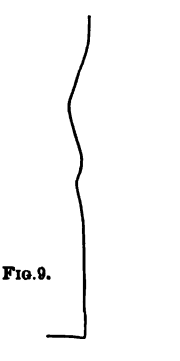

FIG. 9.

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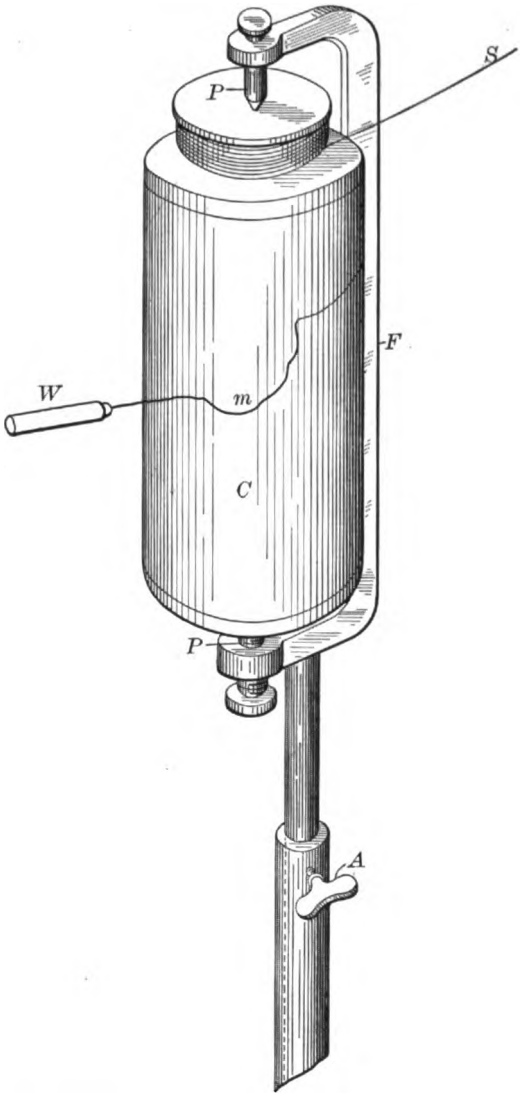

C. Cylinder covered with smoked paper.
F. Frame. *W.* Wire attached to pile.
P. Pivots. *m.* Mark on paper.
S. String. *A.* Adjusting screw for support.

FIG. 11.

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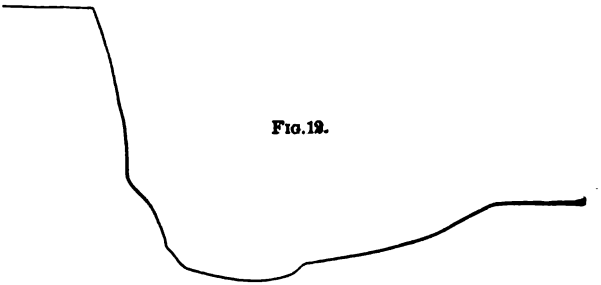

Fig. 12.

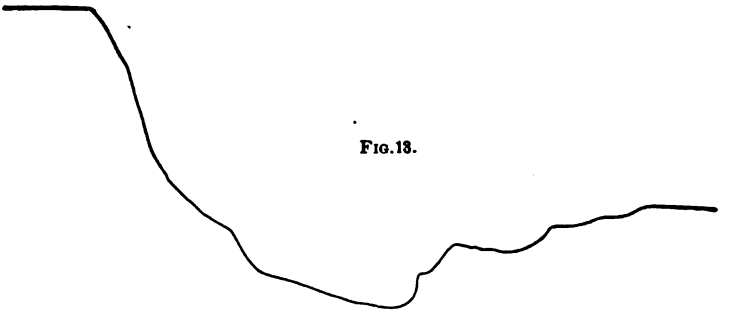

Fig. 13.

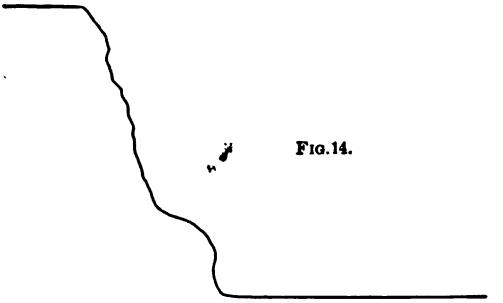

Fig. 14.

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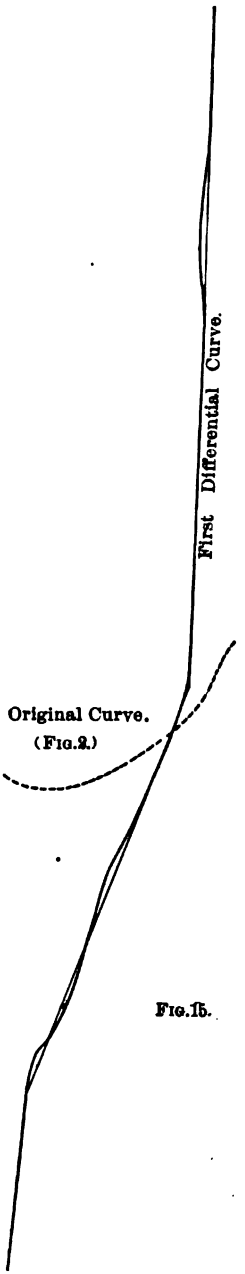

Fig.15.

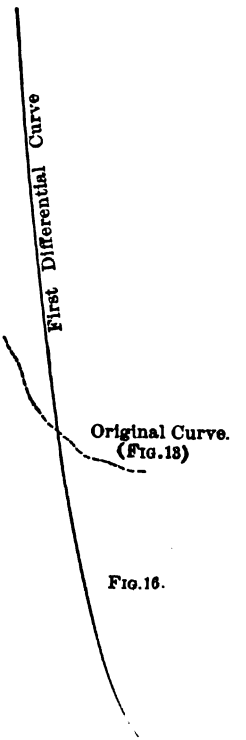

Fig.16.

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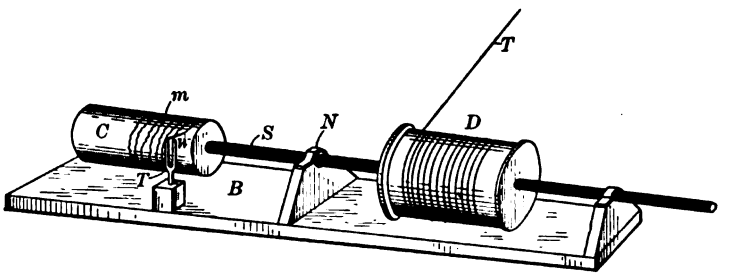
As far as can be seen from the diagrams, the value of a, also, is equal to $\frac{1}{2}$. The diagrams also prove beyond a doubt that the hammer remains in contact with the pile until its motion has entirely ceased. The initial velocity, u_o, of the writer's formula, which is that of the pile itself as distinguished from that of the hammer, or hammer and pile combined, is then equal to v, which is that of the hammer and pile at the instant they move together after all compression has taken place. Therefore, in the writer's formula the quantities r, K and y have the values 1, 1 and 0, respectively. Also, M_h is not equal to zero in the latter portion of this investigation, but must be included in the formula.

RECORDING DEVICE FOR FALL OF HAMMER.



C. Cylinder, covered with smoked paper.
T. String attached to hammer.
D. Drum acting as reel.
B. Base.
S. Threaded axle.
N. Fixed nut.
T. Tuning fork.
m. Mark.
W. Wire on tuning fork.

FIG. 17.

Few authors seem to think that the actual velocity with which the hammer falls differs materially from its theoretical value. To determine this point, two drums were mounted upon the same axle, which could be given a lateral motion by having a threaded portion pass through a fixed nut (Fig. 17). A tuning fork, with a wire soldered to one prong, was made to mark on the smoked surface of one drum as the whole was made to revolve by the unwinding of the string from the other drum. The free end of the string was fastened to a pile-driver

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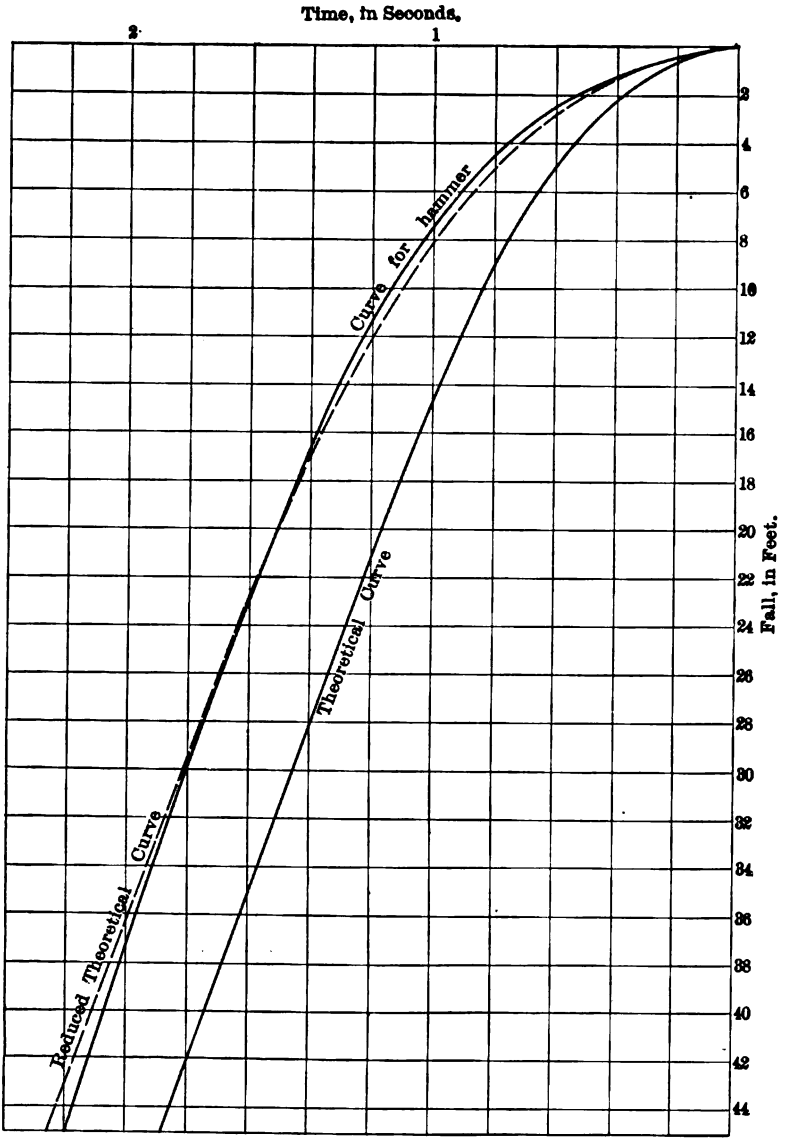

Fig. 18.

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hammer which was allowed to fall in the usual way while the observations were taken. What is considered a typical curve of velocity is shown in Fig. 18, together with the theoretical one. It will be seen from the figure that, after the hammer has fallen about 10 ft. the curves are very nearly parallel. If $2g$ were made $1.15g$, the dotted curve is the one obtained, which is seen to agree very closely with the actual curve obtained experimentally. It may be assumed, then, that the quantities b, m and x of the writer's formula should be $1.15g$, $\frac{1}{2}$ and zero, respectively.

In the discussion of Mr. Crowell's paper by G. B. Nicholson,* M. Am. Soc. C. E., the following examples of the friction suffered by a hammer falling with the usual rope attached are given:

Pile No.	Penetration in Feet—Free fall.	Penetration in Feet—Rope attached.
1	0.7	0.5
2	0.9	0.7
3	0.4	0.32

Upon the assumption that the penetrations are proportional to the velocity of the hammer, the values of b, in the writer's formula, can be computed. These are 1.02, 1.21 and 1.28, respectively, and they average 1.15, which is the value already assumed.

The only other quantity concerning which observations could be made is v. Its value, however, is so very uncertain, in any given case, and so subject to variation, that its consideration will be deferred a little.

The writer's formula, with the above-mentioned substitutions made, is as follows, letting also $v' = \frac{2v}{W_h 1.15h}$:

$$F = \frac{-p}{\left(\frac{cL}{se} + \frac{c'L'}{s'e'}\right)} + \frac{1}{\frac{cL}{se} + \frac{c'L'}{s'e'}} \times \sqrt{p^2 + \left(\frac{cL}{se} + \frac{c'L'}{s'e'}\right)(R_w - v')W_h \times 1.15h}.$$

Practical considerations make it virtually impossible to measure accurately a penetration of less than $\frac{1}{8}$ in., and it is believed that in making actual determiuations it should never be undertaken. It is equally troublesome to obtain any result which can be guaranteed as being within $\frac{1}{8}$ in. of being exact. With a total penetration as large as

* *Transactions*, Am. Soc. C. E., Vol. xxvii, page 172.

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4 ins. (which is seldom observed), a variation of $\frac{1}{8}$ in. would make this penetration liable to 3% error. Assuming the following approximate values for the other quantities in the formula: $c = \frac{1}{2}$, $c' = \frac{1}{2}$, $e = 1\,600\,000$, $e' = 17\,000\,000$, $L = 700$, $L' = 40$, $s = 75$, $s' = 288$, $W_p = 2\,000$, $W_h = 3\,000$, $h = 180$, $W_g = 1\,000$, $v' = 2\%$, we obtain for F, in terms of p,

$$F = 479\,065\,(\sqrt{p^2 + 0.621} - p).$$

Differentiating this expression with respect to p, we observe that the value F changes 8 623 times as fast as p when the latter is 4, and 102 040 times as fast when $p = 1$; and, consequently, an error of 3% in the observed value of p will involve the value of F in at least 3 1% of error in the first case and 23% in the last case. Consequently, any quantities or sets of quantities in the formula which will not change F by 3% when neglected, can, to good advantage, be dropped from further consideration. Further, the liability to error is so enormous with small penetrations that no penetration should be trusted much less than 1 in., and no formula can be guaranteed within a reasonable percentage of error for less penetrations.

The variation caused by the omission of the factors which increase the compression of the pile and hammer are the first which suggest themselves for investigation.

Let $A = \frac{cL}{se} + \frac{c'}{s'}\frac{L'}{e'}$, which can have a value between 0.0000058018 for long piles of soft wood driven by a large, soft, iron hammer, and 0.00000140302 for short, hardwood piles, and a light, tough, iron hammer. Assuming $W_p = 2\,000$, $W_h = 3\,000$, $h = 180$, $W_g = 1\,000$, $v' = 2\%$, $p = 1$, we obtain for the relation between F and A.

$$A F^2 + 2F = 298\,080.$$

If $A =$ zero, $F = 149\,040$; and if A is equal to its maximum value given above, $F =$ about 112 034. This is an extreme variation, in extraordinary cases, of 33 per cent. With a value of 0.000002083, which is upon the assumptions of the last paragraph, $F = 131\,000$, and there is a variation of 17 per cent. Extreme variation in the hammer is found to produce only $\frac{1}{100}$ of 1% variation, so that the quantities c', L', s' and e' may be neglected, simply assuming that the value of $\frac{c'}{s'}\frac{L'}{e'}$ is zero.

In place of $\frac{cL}{se}$, use C, and the value of F now is:

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$$F = \frac{-p}{C} + \frac{1}{C}\sqrt{p^2 + 1.15\,C\,W_h\,h\,(R_w - v')}.$$

The matter of a proper value to be assigned to the v' of this formula is one largely of conjecture. Many authors say that their formulas are to be applied only to a pile having a head firm and free from all brooming. A strict conformity to this dictum would require that practically every pile to be tested should have its head sawed or adzed off just before the test blow is struck. This is almost always impracticable, and thus some value should be assigned to this quantity. From the record of the pile driven by a Nasmyth pile-driver,* by D. J. Whittemore, Past-President, Am. Soc. C. E., it would appear that about 52%, only, of the available energy was actually consumed, on the average, in driving the pile. In driving the pile from the twelfth to the twenty-second foot of penetration, 4 682 blows were struck, or an average of 468 per foot. Immediately after adzing off the head, each of two different times, only 275 and 213 blows, respectively, were required to drive the pile the next foot. Averaging these two would give only 244, which would have been required under first-class conditions. This affords the means for arriving at the above result. The loss in this case is considered excessive.

Professor Franz Kreuter, of Munich, in 1896 presented a paper† in which he showed how the total lost energy could be found by two sets of observations on a pile. Upon the assumption that the loss of energy is the same for falls of hammer not very widely varying, or is proportional to the same, and also that the supporting power is not dependent upon the fall, the value of v', in the writer's formula, can be computed from the two following equations:

$$F = \frac{-p}{C} + \frac{1}{C}\sqrt{p^2 + 1.15\,C\,W_h\,h\,(R_w - v')}, \text{ and}$$

$$F = \frac{-p'}{C} + \frac{1}{C}\sqrt{p'^2 + 1.15\,C\,W_h\,h'\,(R_w - v')}.$$

Observations were made for quite a number of piles, and the corresponding computations of v' made. They were found to vary largely, but v' did not usually exceed 5%, and remained near 2% in most cases where the piles were sound and well driven.

Substituting, as before, values in the last found formula, and letting $v' =$ zero, F then equals 134 400. If $v' = 5\%$, F equals

* *Transactions*, Am. Soc. C. E., Vol. xii, p. 441.

† *Minutes of Proceedings*, Inst. C. E., Vol. cxxiv, Pt. ii.

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124 800—a considerable variation. Without making just such observations as the foregoing, and reducing them, it is absolutely impossible to judge of the size of v'. It is needless to say that such computations are exceedingly irksome, and, according to modern practice, carrying them out would be deemed a needless refinement. When, in

DIAGRAM FOR DETERMINATION OF ENERGY LOST.

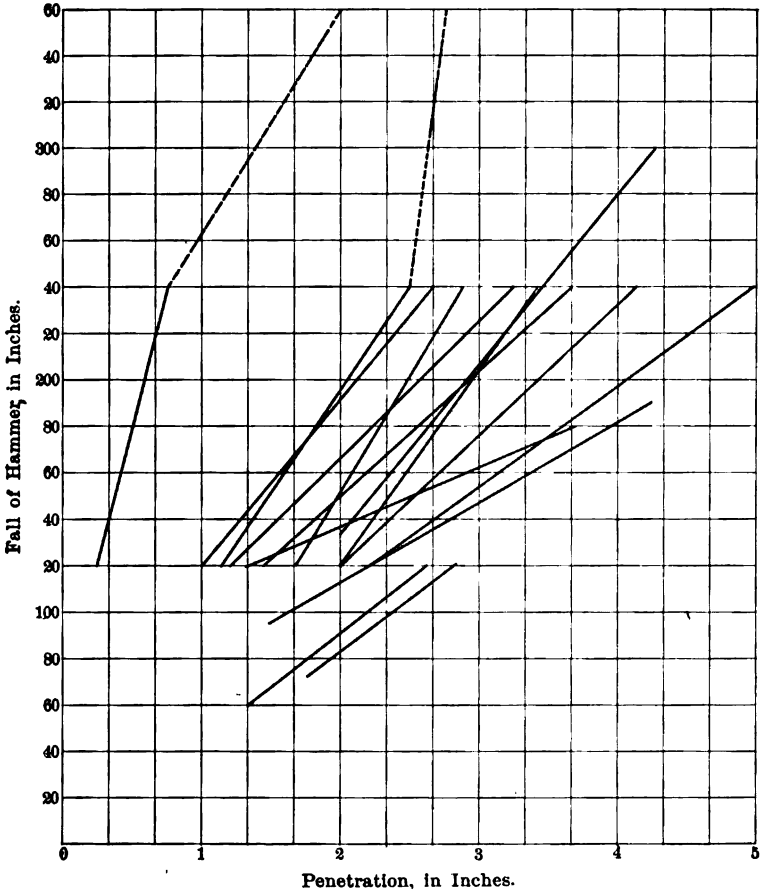

FIG. 19.

the above example, $v' = 2\%$, $F = 130\,560$. A variation of v', 1% either way would involve F in a change of 1.4 per cent. To eliminate the quantity C from the formula, another value of v' might be obtained from two equations in which C had been omitted, so that the value of v' thus obtained should include losses due to

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compression of the pile as well as heating and crushing of its head. The same observations used for v' above were used, but results were obtained graphically instead of analytically, as follows: Heights of fall were plotted as ordinates and penetrations as abscissas, and the line connecting the two points thus determined for each pile was extended to intersect the ordinate axis. This point would show what approximate fall was required to overcome all losses, and its ratio to the average fall in each case would give the value of v' required. Of course, these varied greatly, but averaged less than 10%, even with

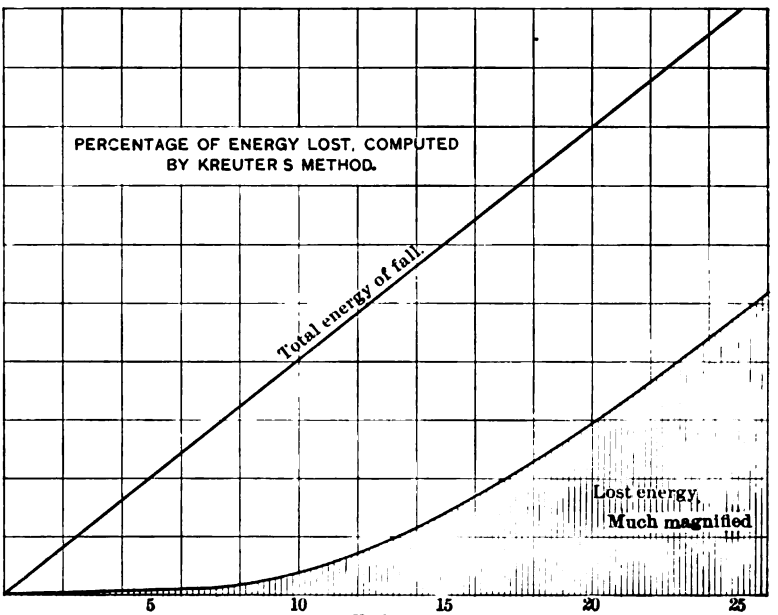

FIG. 20.

some very badly broomed piles. A few plotted observations are shown in Fig. 19. In the cases of the piles driven by the Nasmyth pile-driver and the one shown in Fig. 3, the losses are 48 and 2%, respectively. A very interesting result, shown in Fig. 20, was obtained by computing approximately the percentage of lost energy due to all causes at the different observed falls. It there appears that the loss of energy increases with the fall. The writer's observations tend to show that the quantities involving the compression

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in the pile can be neglected, and their effect compensated for when the piles are sound and well driven, if we make $v' = 2\%$ in the formula. On the other hand, the formula is liable to a 20% error, with poorly driven piles and falls materially less than 15 ft., at which point the value of b is nearly $1.15\ g$.

Making these substitutions, gives for F

$$F = \frac{0.575\ W_h\ h\ (R_w - 0.02)}{p}.$$

The only remaining unknown quantity is R_w. From McAlpine's experiments at the Brooklyn Navy Yard, he concludes* "that, by adding to the weight of the ram, the sustaining power of the pile was increased 0.7 to 0.9 of the amount due to the ratio of the augmented weight of the ram." No experiments were made by the writer concerning this point, as R_w also involves the absolutely unknowable quantity W_g. The experiment with the box of sand, and estimates made of the dirt found clinging to piles withdrawn from the earth, convince the writer that with 70-ft. piles, weighing about 2 000 lbs., W_g should be not less than 1 000 lbs. The assumptions which we are finally forced to make if we desire to reduce the formula still further, involve us in variations which may in special cases amount to 33 per cent. Should this happen in combination with other cumulative errors, the final value obtained may be in error by 50 per cent. On the other hand, if a sound, well-driven pile, weighing somewhat less than the hammer, be tested by a fall of a hammer of about 15 ft., and shows a penetration of about 1 in., the writer feels confident that the final formula will give its supporting power immediately after driving, within a probable error of considerably less than 10 per cent.

If we let $R_w = \frac{1}{2}$, F then is $F = 0.276\ \frac{W_h\ h}{p}$, where h and p are in inches and W_h in pounds.

If it is desired to have h given in feet, the formula becomes $F = \frac{10\ W_h\ H}{3\ p}$, or ten times hammer, times height, divided by three times penetration.

It yet remains to show the relation of this formula to those of other authors. In making this comparison it is to be remembered, however, that this formula was not built to give accurate results under any and all conditions.

* *Minutes of Proceedings*, Inst. C. E., vol. xxvii, p. 8.

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TABLE No. 1.—TESTS OF SUPPORTING POWER OF PILES BY LOADING.

LOCALITY.	Weight of hammer, in pounds.	Hammer fall, in feet.	Penetration of pile, in inches.	SUPPORTING POWER, IN POUNDS.	
				Estimated or observed.	Computed.
Philadelphia	1 600	36	18	14 560	10 666
Mississippi	1 600	25	3	22 400*	44 444
Perth Amboy	1 700	25	2	> 44 800	71 000
Proctorsville	910	5	0.35	62 500	43 000
Brooklyn	2 240	30	0.2	>224 000	1 234 000
Lake Ponchartrain	2 500	30	12	> 22 400	21 000
Aquia Creek	2 000	2	6.50	2 000	2 052
Dordrecht	2 205	25	0.375	13 440(?)	49 020
Buffalo	1 900	29	1.5	75 000	122 000
Fort Delaware	1 900	6	1	15 000	38 000
Boston†	1 710	10	0.7	76 000	81 000

Fig. 21 shows the relationship which exists with other formulas for 3 000, 2 000 and 1 000-lb. hammers and a 15-ft. fall. Table No. 1 shows the actual supporting power of piles (selected from Tables A and B,‡ referred to in connection with Mr. Trautwine's compilation), and others, such as can be used with this formula, in relation to the results indicated by it. Comparison will show that this formula is far closer than any other except Trautwine's, and in the cases selected, and given in Table No. 1, it is nearer than his in seven out of eleven.

It is recommended that in making tests for the supporting power of piles, a standard fall of hammer be adopted and specified for making all determinations. It is recommended that 15 ft. be adopted, and this is done for the following reasons:

(*a*) This height of fall produces a good, observable penetration with any but very light hammers, or for piles in extremely compact soils.

(*b*) The penetration is not excessive for any but very heavy hammers or for piles in very light soils.

(*c*) All frames are large enough to afford this fall.

(*d*) The lost energy is comparatively small.

(*e*) Nearly all formulas give nearly the same values through this region of variation.

(*f*) The writer's formula is especially built for this fall.

Finally, a specification similar to the following, in its main features, is especially recommended:

* From locomotives in use.

† *Journal of the Association of Engineering Societies*, vol. xx. p. 200.

‡ *Transactions*, Am. Soc. C. E., Vol. xxvii, pp. 148–151.

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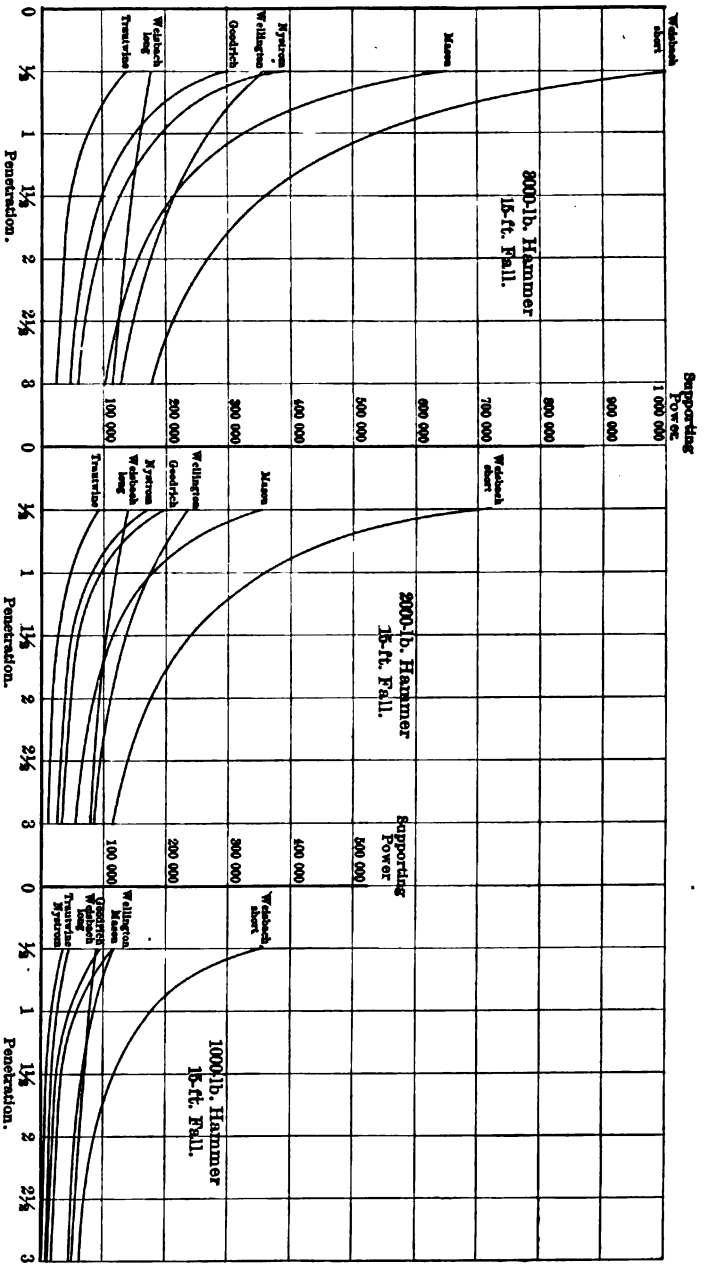
3000-lb. Hammer
15-ft. Fall.
2000-lb. Hammer
15-ft. Fall.
1000-lb. Hammer
15-ft. Fall.
Supporting Power.
Penetration.
Weisbach short
Weisbach long
Trautwine
Nystrom
Wellington
Goodrich
Mason
1 000 000
900 000
800 000
700 000
600 000
500 000
400 000
300 000
200 000
100 000
0
½
1
1½
2
2½
3

FIG. 21.

"Piles shall be driven to such depths that the last blow of a 3 000-lb. hammer freely falling 15 ft. shall not produce a penetration greater than 1 in., or an equivalent penetration directly proportional to the weight of the hammer."

It is believed that such piles will support an ultimate load within 10% of 75 tons; and that designers can more easily determine the necessary pile spacing and the most desirable factor of safety to be used in individual cases, and make the pile-drivers follow a standard specification, than otherwise.

In securing data and checking results, the writer has been assisted materially by P. J. Cleaver, Jun. Am. Soc. C. E., who has acted as Assistant Engineer at the New York Navy Yard for several years.

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DISCUSSION.

E. SHERMAN GOULD, M. Am. Soc. C. E. (by letter).—This paper is valuable in that it exhausts completely the mathematical side of the question of pile-driving, which comes up at intervals with greater or less profit to the profession. No such thorough analysis as is given by the author can ever be deemed useless, because it either establishes a rational formula or demonstrates that none such can exist. It appears to the writer that this paper proves the latter proposition. Mr. Gould.

As a practical art, pile-driving depends wholly and exclusively upon practical experience. Mathematics has nothing whatever to do with it. At most, practice may utilize, under great reserve, some approved empirical formula, which is indeed only the embodiment and concise expression of practical experience. This is abundantly proved by the fact that the practice of pile-driving has been carried to a high degree of perfection, while the science is not yet established.

In the every-day practice of ordinary pile-driving on land and water, it is known that yellow pine piles, 10 to 15 ins. in diameter at the butt, driven to a practical refusal, or until they "fetch up," with a hammer weighing from 2 000 to 4 000 lbs., with a fall of from 5 to 20 ft., all according to circumstances, give satisfactory results; the only remaining question being how many shall be driven in the given area. In all ordinary work, considerations of continuity of bearing result in spacing the piles so close together that their bearing capacity, as determined by any known formula, vastly exceeds the weight to be placed upon them. When any doubt exists as to the probable length of piles required, recourse is had to an actual test. If piles of great length are to be driven in uncertain ground, to support an important structure, the best available expert advice should be invoked, rather than the best mathematical talent.

The only obstacle to reducing any engineering proposition to a rational formula is uncertainty as to data. If all the data are known, to a certainty, the problem falls inevitably within the iron grasp of analysis, from which it cannot escape, and to which it must yield its secret. In pile-driving, data are conspicuously lacking. Of course the weight of the hammer, the height of fall, etc., are known, but all these factors are affected by unknowable and varying coefficients. What are the conditions under which a 4 000-lb. hammer, falling 20 ft., strikes the head of a pile, 15 ins. at the butt? In the first place, the force of the blow is entirely under the control of the man who has one hand on the throttle, and the other on the drum-brake. How lightly he can tap the pile! He can almost crack a hickory nut on it without injuring the kernel. But, admitting that he acts in good faith, and "lets go altogether," the falling weight must overhaul the rope and revolve the drum in its descent, and also overcome the friction of the

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Mr. Gould. leaders. Several men will be keeping the pile in position by jamming handspikes between it and the leaders, and by hauling it in by the headlines on the winch. There is also the brooming of the head of the pile, above the ring, upon which the hammer cushions itself to a greater or less extent. All these resistances are to be added to that offered to the pile by the substance through which it is being driven, and these circumstances—and many more—destroy all hope of a rational formula.

In the writer's opinion, no formula can be applied, and no intelligent guess made as to the bearing capacity of a pile, unless it is driven to a practical refusal of, say, 1 inch. The best way is to drive the pile down rapidly until it either refuses a high fall, or has gone down nearly to grade, and, in the latter case, to reduce the fall till refusal is exhibited, and then, if desired, apply the formula. He believes, also, that refusal can be estimated by eye without actual measurement; it is very noticeable when penetration becomes labored, and the pile is beginning to fetch up. All attempts at refinement of measurements of the fall and penetration have the fatal defect of retarding the rapidity with which the pile is sent home. When piles are driven down to a hard substratum there is, of course, no difficulty in telling when they are home, and all hammering should then be stopped.

Another element which makes for safety, but which baffles calculation, is the clinging action of the material through which the pile is driven, and which action is set up immediately after it has been allowed to come to rest. It is often impossible to draw a defective pile even a very short time after it has been driven, unless a few blows be given by the hammer to start it, when it may come up very easily. A pile which has gone down readily to-day may utterly refuse all further penetration under the same hammer and fall to-morrow; for this reason, driving should be continuous, till the pile is home.

The writer observes with some surprise that the author makes no mention of the paper* presented by Charles H. Haswell, M. Am. Soc. C. E., which gave rise to an instructive discussion. In this paper Mr. Haswell gave a formula, which, admitting a set of $\frac{1}{2}$ in., and a factor of safety of 6, reduces to

$$L = 5\ W\sqrt{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which L = safe load, and W = weight of hammer, both in the same unit, and h = fall, in feet. Wellington's formula, given in the same paper, admitting a set of 1 in., reduces to

$$L = W\,h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

the nomenclature remaining the same. If a set of $\frac{1}{2}$ in. be admitted, the same as in Equation (1), then Equation (2) reduces to

$$L = 1.33\ W\,h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

* *Transactions*, Am. Soc. C. E., vol. xlii., p. 267.

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For falls of between 10 and 20 ft., Equations (1) and (3) give nearly equal values of L. Either is serviceable under average conditions. Mr. Gould.

In thus exalting the part which the trained judgment plays in the art of pile-driving, the writer does not wish to detract in the least from the high degree of analytical ability displayed by the author.

HORACE J. HOWE, M. Am. Soc. C. E.—Mr. Goodrich has referred to the speaker's paper on piles and pile-driving,* which went into the subject from a historical and experimental standpoint. Mr. Howe.

Since that time few detailed tests have come to the speaker's notice, the most interesting being those at the Annapolis Naval Academy during the construction of the sea-wall, a year or more ago, and partially described by J. P. Carlin,† Jun. Am. Soc. C. E., of the contracting firm which did this work. He states that five piles were loaded singly, to the ultimate, and that the results were compared with the Wellington (*Engineering News*) formula, as shown in Table No. 2.

TABLE No. 2.

Pile.	Safe load, as per formula.	Actual ultimate load.	Remarks.
1	38 000	75 000	Point in mud and sand.
2	40 000	85 000	" " "
3	32 000	34 000	Point in mud.
4	33 800	38 000	Pile in sand.
5	50 000	110 000	" "

The third test pile was entirely in mud, and the factor of safety was assumed as one, in designing the sea-wall at that point. It seems, however, that both the formula and the load actually supported gave figures which were too small, and that the wall settled for 100 ft. in length.

Mr. Carlin can doubtless supply further details.

In the paper above referred to, after a somewhat exhaustive review, the speaker called attention to the fact that single test piles, whether separate or taken from a cluster, are inadequate; and that a test is accurately adequate only when it fulfills all of the subsequent designed conditions of loading, and covers a sufficient area and lasts for a sufficient period of time. The failure of this wall is testimony as to the soundness of these conclusions.

Half a century ago, at Fort Delaware, Major Sanders made two sets of experiments on clusters of four piles each, and extended his observations on the same for some years; afterward evolving his well-known formula. Recent reports (1897) as to the masonry indicate no settlement, and the conclusion is that, under exactly those conditions, Sanders' formula is to be considered applicable.

* *Journal of the Association of Engineering Societies*, April, 1898.

† *The Engineering Record*, May 11th, 1901.

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Mr. Howe. Looked at from the standpoint of his contribution to the subject, it may be that a formula is not such a forlorn hope, after all, as some have been in the habit of thinking; and that in expert hands it may attain positive value.

It is hoped that the profession may hear further from Mr. Goodrich along these lines, and that he will extend his clever observations and give us and himself the satisfaction of checking, or, if necessary, revising his mathematics.

Mr. Carlin. JOSEPH P. CARLIN, Jun. Am. Soc. C. E. (by letter).—In an article* by the writer a description is given of some tests of piles, in connection with the construction of a sea-wall at the Annapolis Naval Academy, of which the following is an abstract:

Tests Nos. 1 and 2 were on the same pile, although the second test was on the pile after it had been driven 6 ft. further.

The original borings had indicated hard bottom at a uniform depth of 40 ft. below the river bed, excepting at one point, where they showed a depth of mud of 70 ft., with 7 ft. more of mud and sand before hard bottom was reached. This point, therefore, was selected for the site of the tests.

The usual data were taken during the driving. Then the head of the pile was squared and dowelled, and a timber frame, slung by means of wire-rope lashings, was securely fastened about the 3-in. steel dowel. To prevent lateral swaying, four guys were run out from the head, and their ends made fast to powerful kedge anchors. These guys were very nearly horizontal. The pile was then loaded with anchor chains and shot, each shot having been weighed separately and tagged. The frame weighed more than 9 tons.

The sea-wall, along the site of the tests, has been finished. For about 200 ft. of wall the piles were in sand, and there has been no settlement. The next 100 ft., however, was in mud (Test No. 3), and there has been a settlement of 10 ins., which was subsequently arrested by driving additional re-enforcement piles of greater length, and blocking up from them under the wall.

In conclusion, the writer believes that, while the first two tests indicated fair results, the third demonstrated that, considered independently, the Wellington formula, or any other, is practically useless. The fourth and fifth tests give a striking lack of uniformity in the results.

It seems to be very necessary that the conditions surrounding the pile be the same as will be the case, ultimately, in the permanent structure; and that this test pile be observed during a period as long as the opportunity will permit, certainly not less than two or three months. If it be not possible to apply this time test, a carrying-capacity experiment would be no more satisfactory than the resort to the ordinary penetration records, together with an intelligent investi-

* *The Engineering Record*, May 11th, 1901.

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gation of borings, both governed by experience with variable conditions. Mr. Carlin.

ERNEST P. GOODRICH, Jun. Am. Soc. C. E. (by letter).—No pile formula can give more than an approximation to the supporting power of the special pile observed, and only at the time of driving; but, with an intimate knowledge of the soil conditions, a good pile formula becomes of value, and considerable money often can be saved, at the time of driving, through its proper application. This is where the science of pile driving can influence the art. Mr. Goodrich.

TABLE No. 3.—VARIATIONS IN SUPPORTING POWER, FROM VARYING ASSUMPTIONS.

v'	R_W	Formula.	F for p equals 1.	F for p equals $\frac{1}{4}$.	F for p equals 4.	Percentage of Error.
0	.....	$0.287\,\frac{Wh}{p}$	156 000	.....	.....	4.3
2	.....	$0.276\,\frac{Wh}{p}$	149 040	.....	.....	0.0
5	.....	$0.256\,\frac{Wh}{p}$	138 000	.....	.....	6.0
10	.....	$0.230\,\frac{Wh}{p}$	124 000	.....	.....	11
20	.....	$0.173\,\frac{Wh}{p}$	93 500	.....	.....	37
2	$\frac{1}{8}$	$0.276\,\frac{Wh}{p}$	149 040	.....	.....	0.0
2	$\frac{1}{2}$	$0.204\,\frac{Wh}{p}$	110 160	.....	.....	26
2	$\frac{1}{4}$	$0.132\,\frac{Wh}{p}$	71 280	.....	.....	52
2	.....	$3.312\,\frac{WH}{p}$	149 040	.....	.....	0.0
2	.....	$3.312\,\frac{WH}{p}$	.....	596 160	.....	0.0
2	.....	$3.312\,\frac{WH}{p}$	.....	.....	37 260	0.0
2	.....	$10\,\frac{WH}{3p}$	150 000	.....	.....	0.6
2	.....	$10\,\frac{WH}{3p}$	.....	600 000	.....	0.6
2	.....	$10\,\frac{WH}{3p}$	.....	.....	37 500	0.6
0	.....	$\frac{10\,WH}{3p+\frac{1}{10}}$	145 100	.....	.....	2
0	.....	$\frac{10\,WH}{3p+\frac{1}{10}}$	.....	529 411	.....	11
0	.....	$\frac{10\,WH}{3p+\frac{1}{10}}$	.....	.....	37 190	0.1
0	.....	$\frac{3.444\,WH}{p+0.04}$	148 058	.....	.....	0.6
0	.....	$\frac{3.444\,WH}{p+0.04}$	.....	534 414	.....	10
0	.....	$\frac{3.444\,WH}{p+0.04}$	.....	.....	38 360	3.0

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Mr. Goodrich.

Of course, it is eminently better to test piles under the actual conditions to be encountered; but this is almost invariably impossible, the few actual tests of even single piles showing this conclusively.

After the accumulation of whatever evidence and experience has come down to us, we seem to be justified in assuming that Sanders' formula, as it is usually known, when applied to penetrations of from $\frac{1}{8}$ to 1 in., comes sufficiently near the truth, with perhaps the need of a slight change in his constant.

Two other formulas, somewhat different in character in that they involve falls of at least two different heights, may be added to the list for reference:

Haagsma: $\frac{h - h'}{p - p'} \times \frac{W_h}{(W_h + W_p)}$

Morrison: } $\frac{h - h'}{p - p'} \times W_h$.
Kreuter: }

Haswell: constant $\times W_h \sqrt{h}$

McAlpine: $30 [W_h + (0.228 \sqrt{h} - 1) 2240]$.

Haswell's formula is based on a penetration of $\frac{1}{8}$ in. only, and hence is hardly comparable with the others in that it does not involve a variable p. In this respect, it is like that of McAlpine, who also has $\sqrt{h}$. Their observations and conclusions, which make the supporting power vary with the square root of the fall, are at variance with the work of the writer and of all other observers.

To show the possibility of wide variation, even with a most carefully prepared formula, for small variation in conditions, Table No. 3 has been prepared. It is believed to be self-explanatory. In computing F, W_h has been taken as 3 000, h as 180, H as 15, p various, v' various and R_W various.

The Annapolis tests, which had entirely escaped the attention of the writer, are of great interest. The actual load and the load computed by the writer's final formula are shown in Table No. 4.

TABLE No. 4.

Number.	Length.	Point.	Butt.	Hammer.	Fall.	Penetration.	Actual load.	Formula: $\frac{10 W H}{3 p}$	Nature of Soil.
1...	91	7	22	2 800	22	1⅛	75 000	96 500	Water, 12 ft.; mud, 60 ft.; sand, 6 ft.
2...	91	7	22	2 800	22	1¼	85 090	112 600	Water, 12 ft.; mud, 60 ft.; sand 12 ft.
3...	73	9	18	2 800	33½	3⅜	34 000	67 000	Water, 12 ft.; mud, 61 ft.
4...	80	12	8	2 800	22	2	38 000	84 500	Sand.
5...	82	13	9	2 800	22	1	110 000	168 666	Sand.

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When the actual values of R_w and properly assumed values of v' are used, much closer computed results are found, but, under ordinary circumstances, such refinements are not practicable. Mr. Goodrich.

The peculiar, and apparently erratic, variation in the results can be readily and satisfactorily explained by the soil conditions, but strongly go to prove that a pile formula alone, without other knowledge, is indeed a poor crutch.

The matter of a proper factor of safety is believed to be one which must be settled specially by each engineer, for each piece of work in hand, from known soil conditions and the uses to which the foundation will be put.

It is well to note that a pile will fail by crushing with a load of approximately 6 000 lbs. multiplied by the square of the diameter, in inches; and loads greater than given by this, where found by pile formulas, should be discarded.

A series of experiments somewhat similar to those made by the writer and illustrated in Figs. 2 to 14, was carried out by Mr. J. M. Heppel, in England, some years previous to 1867, but his description* is too meager to afford much information.

The writer begs to acknowledge his appreciation of the kind remarks made by those discussing his work.

* *Minutes of Proceedings*, Inst. C. E., Vol. xxvii, p. 42.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 922.

ADDRESS AT THE ANNUAL CONVENTION AT WASHINGTON, D. C., MAY 20TH, 1902.

THE FIRST FIFTY YEARS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS: 1852–1902.

By CHAS. WARREN HUNT, M. Am. Soc. C. E.

At the request of the Board of Direction the speaker has prepared for presentation to this meeting a summary of the history of the Society during the first fifty years of its existence, which period will be completed before the next general meeting.

Perhaps the best way of introducing the subject will be to read the minutes of the first meeting of the Society, as follows:

"On the evening of the 5th of November, 1852, a number of Civil Engineers and Architects convened at the office of the Croton Aqueduct Department, in the City of New York, agreeably to a circular issued on the 23d day of October by several members of the professions to their professional brethren in this city and vicinity, the following gentlemen being present: Julius W. Adams, New York; James Laurie, New York; Thomas A. Emmet, New York; J. W. Ayres, New York; Edward Gardiner, New York; W. H. Talcott, New Jersey; Robert B. Gorsuch, New York; W. H. Morrell, New York; W. H. Sidell, New York; S. S. Post, Owego, Tioga Co., N. Y.; Alfred W. Craven, New York; G. S. Greene, Albany.

"The meeting was called to order by A. W. Craven, on whose motion Maj. W. H. Morrell was elected to the chair, when, upon the

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motion of James Laurie, T. A. Emmet and R. B. Gorsuch were appointed Secretaries.

"James Laurie then moved that a committee to report a constitution and by-laws be appointed by the chair, which was adopted; and James Laurie, Julius W. Adams and W. H. Sidell were appointed.

"The Committee retired for consultation, and after a short absence reported a draft of a constitution and by-laws, also a copy of the general law of the State of New York authorizing the incorporation of scientific societies, which were severally read to the meeting by J. W. Adams.

"The Constitution was then taken up by sections, discussed, amended and adopted.

"James Laurie presented the following resolution:

"Resolved, That the American Society of Civil Engineers and Architects be incorporated under the act of 1848, Chap. 319 of Laws of New York, which was adopted. It was also

"Resolved, That the members of this Society shall be those who, being by the Constitution eligible, sign the Constitution at the time of its adoption, and those who may be invited by the Board of Direction to attend the first regular meeting, to be held on the first Wednesday in December next, and shall attend or signify their intention to become members on or before that date, and shall pay to the Treasurer their entrance fee and first annual contribution, and all those who shall thereafter become members in conformity with the Constitution.

"A. W. Craven then moved that a committee be appointed to nominate officers, which was adopted.

"S. S. Post moved that the committee be a committee of the whole, whereupon James Laurie offered an amendment that the committee consist of three. The question was first taken on Mr. Laurie's amendment, which was lost; then on Mr. Post's motion, which was carried.

"The meeting then resolved itself into a committee of the whole to nominate officers. The Committee presented the following nominations.

For President - - -	James Laurie,
For Vice-Presidents - -	Edward Gardiner,
" - -	C. W. Copeland,
For Directors - - -	W. H. Morrell,
" - - -	W. H. Sidell,
" - - -	J. W. Adams,
" - - -	J. P. Kirkwood,
" - - -	A. W. Craven,
For Secretary and Treasurer	Robert B. Gorsuch.

"The meeting then proceeded to ballot, when the above-named officers were unanimously elected.

"James Laurie presented the following resolution:

"Resolved, That the Board of Direction be authorized to issue a

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circular asking the aid and co-operation of those eligible as members in sustaining this Society, and to have the Constitution, By-Laws and Act of Incorporation printed for the use of the members, which was adopted.

"A vote of thanks was then tendered to the Chairman and Secretaries, and the meeting then adjourned until the first Wednesday in December.

"R. B. GORSUCH, *Secretary.*"

"JAMES LAURIE, *President.*"

Of those who attended this meeting, and thus became the founders of the Society, the only one known to be alive is Robert B. Gorsuch, its first Secretary and Treasurer, who is still in the active practice of the profession in the City of Mexico.

The objects of the Society, as incorporated in the Constitution which was adopted at that meeting, and issued in a circular to engineers five days later, were:

"The professional improvement of its members, the encouragement of social intercourse among men of practical science, the advancement of engineering in its several branches, and of architecture, and the establishment of a central point of reference and union for its members." And—this circular continues—"Among the means to be employed for attaining these ends shall be periodical meetings for the reading of professional papers, and the discussion of scientific subjects, the foundation of a library, the collection of maps, drawings and models, and the publication of such parts of the proceedings as may be deemed expedient."

At the second meeting, which was held December 1st, 1852, By-Laws were proposed and were "read, discussed, amended and adopted."

Eight meetings were held during 1853, at which the average attendance was six, and only fourteen individuals, all told, came to these meetings. At the end of the first year, the total membership was forty-eight, the total receipts $700, and the total expenditures, $115.22.

In 1854 but six meetings were held, at which the average attendance was less than that of the previous year. The total receipts of that year also decreased to $150, and expenditures to $32.02.

It may be of interest to note that the first subject for professional discussion was "The Relief of Broadway," and that Mr. James Laurie, President, at the meeting of January 5th, 1853, described a plan for placing railway tracks above the level of the street, the subject being discussed by the members present.

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At one of the meetings in 1854, the death of John Hampson, Esq., Engineer, New Orleans and Carrollton Railroad, was announced, and as matter of interest it was stated that "Mr. Hampson aided in the construction of (as a Machinist) and ran as Engineman the first locomotive engine built in America, 'Brother Jonathan,' which, by contract, was to weigh, with wood, water, and three men, but 3½ tons."

The first formal paper was presented to the Society on March 2d, 1855. It was by William H. Talcott, and was entitled "Results of Some Experiments on the Strength of Cast Iron."

It appears that at this early stage the founders of the Society had great difficulty in interesting in it engineers who lived at a distance, and early in January, 1855, one of its most active promoters, Mr. James O. Morse, who a month previous had been elected Secretary and Treasurer, tendered his resignation of the former office to take effect as soon as his successor was appointed. In 1867, inasmuch as no such appointment had been made (and for more than twelve years no meetings of the Society held), Mr. Morse was still the Secretary, and became active in the reorganization. He served continuously as Treasurer of the Society for twenty-one years.

On October 2d, 1867, pursuant to the call of Mr. James Laurie, President, a special meeting of the Society was held at the office of C. W. Copeland, 171 Broadway, five of those who were present at the first meeting in 1852, and five others, being present. The minutes of the last meeting of the Society were read and approved, and steps were taken to revive interest in the organization. It was determined to establish a headquarters as soon as possible, and this was accomplished in less than a month.

The Annual Meeting of November 6th, 1867, in the Chamber of Commerce Building, 63 William Street, was the first held in rooms devoted exclusively to the purposes of the Society. It was found at this meeting that Mr. Morse, the Treasurer, having placed the funds of the Society in a Savings Bank, had nearly doubled them, so that there was more than $1 000 in the treasury with which to start. About half of this sum resulted from dividends received by Mr. Laurie, President, in the interim, on five shares of New York Central Railroad stock, with compound interest added.

The following resolution was passed at this meeting:

"*Resolved*, That we tender our thanks to Mr. James Laurie for his faithful services as our President, for his efforts to re-establish and

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reorganize this Society on a basis which gives promise of a successful and useful continuance, and particularly for his care of our funds, to which we are greatly indebted for our present unencumbered and hopeful condition."

Continuously since this reorganization, semi-monthly meetings, except during the summer, have been held on the first and third Wednesdays of each month.

The first publication was the Address of President James P. Kirkwood, delivered December 4th, 1867, but not printed until somewhat later. It constitutes the first pages of Volume I of *Transactions*, which bears the date of 1872. In that address, Mr. Kirkwood points out that the original failure of the Society to interest non-resident members was due to the fact that no place was established to which members "could point as evidence of our being something more than a parchment Society," and that in order to maintain a bond between all classes of members it was necessary that papers should be printed and disseminated to those who could not be present at the meetings.

In March, 1868, the name of the Society was changed to that which it now bears. It was not until 1877, however, that the failure of the officers to carry out the instructions given in 1852 for the incorporation of the Society was discovered, so that, the proper steps being taken, the original incorporation of the Society was under its present name.

It seems needless to dwell here upon the popularity and success, professionally and socially, of the Annual Conventions. The first of these was held in New York City, June 16th, 1869, fifty-five members being present. Five formal papers were presented, and a dinner was served in the Chamber of Commerce Building.

The second and third Conventions were also held in New York City, but the desirability of holding these meetings at widely separated points soon became apparent, and the present system was inaugurated with the fourth Convention, which was held in Chicago, Ill., in June, 1872.

The Annual Report of 1873 calls attention to the growing attendance at the Society meetings, and speaks of the Convention held in Louisville, Ky., May 21st and 22d of that year (29 years ago, almost to a day) as being the largest meeting of civil engineers ever assembled on this continent. The total attendance was seventy-nine, and the rapid growth of the Society is indicated by the fact that there are

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almost as many persons connected with the Society now residing in the District of Columbia as were present at that meeting.

At the end of 1869 the total membership was 160, and since that time the average net yearly increase has been as follows:

For the ten years			1870 to 1879	44.1
"	"	"	1880 to 1889	69.5
"	"	"	1890 to 1899	90.3
"	two	"	1900 to 1901	164.5

The total membership to-day is 2 603.

During the entire life of the Society the proportion of resident and non-resident members has been practically constant—between 75 and 80% of the total membership belonging to the latter class.

Since the first home of the Society was established, as previously stated, in 1867, its headquarters have been moved four times, and each time because more room was necessary for carrying on the work. The first two moves were to temporary quarters, but in 1881, No. 127 East 23d Street, a typical 25-ft. New York City residence, was purchased. This house was enlarged in 1887, but, notwithstanding this, in 1895 more room was needed, not only to accommodate the growing attendance at meetings, but because proper facilities could not be furnished for taking care of the rapidly increasing library. The present property at 220 West 57th Street was accordingly purchased, and the construction of a new house completed two years later. The house, which was opened with appropriate ceremonies November 24th, 1897, has a frontage of 50 ft., and a depth of more than 100 ft., and was designed exclusively for the purposes of the Society. The last report of the Secretary shows that the total assets of the Society above all indebtedness are more than $215 000.

It is interesting also to note that while minor donations for special purposes have been received by the Society, its growth and prosperity have been entirely dependent on its own resources. It has no endowment, and has always been supported by the dues paid by its members.

In 1896 the system of publications was entirely changed, the object being to issue in a regular monthly publication all professional papers in advance of the date set for their discussion, in order to allow to every member opportunity to give his views on any subject presented. The gratifying results of this change have been given recently in detail in the annual reports.

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Since the occupation of the new house, the Library, which now contains more than 40 000 volumes, has been rearranged, classified and thoroughly indexed, and two years ago a complete catalogue was issued. The addition of more than 6 000 volumes during 1901 indicates clearly the rapidity of the growth of such a library when systematic efforts are made to complete its files and bring it up to date.

Many other things interesting from a statistical or historical point of view might be mentioned, but in the limited time which can be allotted at this meeting, it is not possible to do more than allude in a general way to the work of the Society in carrying out the objects for which, with wise forethought, it was founded.

The organization is in no sense a local one, and it has never had any subsidiary branches or been affiliated with other organizations. Its membership is made up of practitioners engaged in all branches of Civil Engineering, the broadest interpretation of that term being used. From the beginning, admission to its privileges has been dependent solely upon professional experience and personal character.

That the Society is beneficial to the profession is evidenced by the eagerness with which membership in it is sought. The reasons for this are apparent, inasmuch as among its objects are to assist the young engineer professionally during the earlier years of his career, and, when he has proved himself worthy, to stamp him as one qualified to direct "the great sources of Power in Nature for the use and convenience of Man."

That its influence is far-reaching is shown by the fact that it has members in 51 of the 54 subdivisions of this Republic, as well as in 31 foreign countries. This foreign membership constitutes 9½% of the total list.

If its policy continues to be as increasingly progressive as it has been in the past in its various lines of usefulness, viz., the standard of Membership, the Publications, the Library, and last, but by no means least, the Meetings and Conventions, where social intercourse "between men of practical science," is encouraged; it must continue to be a potent factor in the material progress of every country to which its membership extends, as well as in the improvement of the status of the individual practitioner, and of the Profession of Civil Engineering.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 923.

ADDRESS AT THE ANNUAL CONVENTION AT WASHINGTON, D. C., MAY 20TH, 1902.

THE ENGINEER OF THE TWENTIETH CENTURY.

By ROBERT MOORE, President, Am. Soc. C. E.

At our last Annual Convention my predecessor addressed you upon Engineering in the Nineteenth Century, and at this meeting our Secretary has given you a sketch of the history of our own Society during its first fifty years, which will have closed before our next annual reunion.

It has seemed to me, therefore, not inappropriate that to-day I should take a glance toward the future, and say something concerning the outlook for Engineering and for our own Society in the new century upon which we have just entered.

It is true that in doing this we must to some extent leave the solid ground of fact, which is the special province of the engineer, and enter upon the insecure and dimly lighted field of possibility and prediction in which the most wary may easily be led astray. On the other hand, it is entirely safe and legitimate to predict the fruit from the seed, and all wise action in the present must be based upon correct views of the results which they will produce. In every field of activity the successful doer is the clear and successful seer. What, in fact, is the whole life of the engineer but an effort to conceive plans and execute works by which the future shall be made to surpass the

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present? So that, as engineers, we may claim the future as part of our field, no less than the present or the past.

What kind of a man, then, will the engineer of the new century be, and what the scope of his work, as compared with the engineer and his work in the century which has just closed?

Before proceeding to consider this question let me say that in speaking of the Engineer I shall use the word in the comprehensive sense of Thomas Tredgold's well-known phrase as one who directs "the great sources of Power in Nature for the use and convenience of Man," a sense which includes every subdivision of this work. Carlyle has rightly said that man is pre-eminently a tool-using animal, one who converts the materials and forces of the external world into instruments of service, thus making Nature his slave instead of his master. The engineer is one who devotes his life to the designing and making of tools.

With this understanding of the word, what shall we say of the engineer of the new century as compared with the engineer of the century which has just closed? In answer, we may, with perfect assurance, say that he will be a man of more comprehensive and thorough training, a better equipped and therefore more serviceable man than his predecessor of the last century.

Until very recent years the engineer as a man of learning, even in his own profession, has not been the peer of the clergyman, the teacher, the lawyer or the physician. In fact, speaking broadly, engineering has not been one of the learned professions. And for this the reasons are not far to seek. As a distinct profession, that of the engineer is much younger than those just mentioned, and, what is even more to the point, the sciences which set forth the laws that govern the materials and forces of Nature and underlie engineering works are all of recent development.

Historically, as we all know, the oldest profession is that of the priesthood. How to ward off evil spirits and propitiate the invisible powers has been one of the earliest studies of every people, and they who knew, or were believed to know, best how these ends might be accomplished were differentiated into a separate class who became the custodians of all the learning then attained by men. The priests were the scholars, the learned men, of their time. As such they were the medicine-men, or physicians, the healing art being at first almost

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wholly a matter of incantation. As conservators of tradition, they became the arbiters in disputed matters, and thus the lawyers and judges. As the only learned men, they were, of course, the first teachers or instructors in letters. In important structural works, the first of which were largely temples, they were the natural leaders and, as such, the first architects. And their early activity in the field of engineering may be seen in the fact that among the Romans—the most practical of nations—the high priests were styled pontiffs or bridge-makers, and the high priest was *pontifex maximus* or the chief bridge builder. And the oldest engineering society was no doubt that of the *Fratres Pontifici*, or the bridge-building brotherhood of Benedictine monks, who, during the Middle Ages, chose as their special work the building and repair of bridges.

How well much of this early work was done many an old cathedral and many an ancient bridge bear abundant witness.

In process of time, however, each of these subordinate fields of knowledge and practice became so extensive and so exacting of time as to be beyond the compass of any one man or class of men, and each gradually became the special field of a distinct profession.

In the order of time, the physician, the lawyer and the teacher—the professor *par excellence*—came first.

Then followed the architect, as the skilled designer of houses, and, last of all, the engineer or the millwright and the skilled maker of roads and bridges.

But the structures which the first engineers were called to build, while they demanded skill and courage and judgment, called for little of the learning found in books. Some practical skill in arithmetic and geometry, and possibly trigonometry, was the limit of the scientific knowledge required. Engineering was an art and not a science, and the engineer was little more than a highly skilled workman. And, as in all the arts, the standard method of instruction was that of apprenticeship in which by word and by example the master slowly transmitted to his pupil the art and mystery of his craft.

James Brindley, builder of the first important English canal, was a cottier's son, and served seven years as a wheelwright's apprentice. George Stephenson, builder of the first successful locomotive and railway, was eighteen years of age before he learned to read. Thomas Telford, builder of roads and bridges, and first President of the Insti-

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tution of Civil Engineers, was the orphaned son of a shepherd, and served a seven years' apprenticeship as a stone mason. Smeaton and Watt were mathematical instrument makers, and both men of considerable training, but, as engineers, were almost wholly self-taught.

In like manner, in our own country, the engineers who in the first half of the nineteenth century built our first canals and railroads—such men as James Geddes, Benjamin Wright, Canvass White, Loammi Baldwin, Nathan Roberts and Jonathan Knight—were all men with no learning beyond that of the elementary schools, and whose whole professional training was gained in the school of experience. Though men of judgment, skill and courage, men of force and of brains, they were not men of science; for beyond mathematics, the higher developments of which were known only to a very few, the sciences which underlie the engineer's work had not yet taken form. Structural mechanics, thermo-dynamics, chemistry and electricity as organized sciences did not exist. But since Carnot, Davy, Faraday, Joule, Rankine, Maxwell, Clausius, and others like them, have done their work, all this is changed, and he who would now enter the field as an engineer, with the slightest hope of success, must have mastered the sciences which the labors of these men have developed. The knowledge which had to suffice for his predecessors is no longer enough.

Out of this new need, and as the shortest road to such mastery, has grown the engineering school, in which these new sciences are especially taught—a growth which has been characteristic of the last half century, and which, during the last few years, has been phenomenal.

As introductory to the engineering school, and as the foundation upon which its work must be laid, there is needed, of course, a thorough grounding in the general principles of the sciences of matter and motion, to wit, elementary mechanics, physics, and, above all, mathematics, or the science of number and measurement. But, in addition to this training in the elements of the mathematical and physical sciences, there is also needed for the proper equipment of the engineer a broad elementary training in the other departments of human knowledge, which are, by contrast, metaphysical, and, as yet, unmathematical. For we cannot know anything correctly, except as we know its limits and its place in the general system of things. Even the place where we stand on the earth cannot be accurately located, except by its co-ordinates of latitude and longitude which define its position with reference to the whole globe. Every

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well-trained professional man, therefore, must have, as part of his equipment, the broad outlines of general knowledge. He must know something of language and literature, of political science and of history, something of what the world's workers have done and of what its thinkers have thought, in order that he may correctly understand and evaluate the knowledge peculiar to his own calling. To know everything about something, which is his business as a professional man, he must also know something about everything.

A broad basis of general knowledge, such as shall put the man in touch with all times and with all men, must, therefore, precede the special knowledge of the technical school. Otherwise, there may be much mastery of detail, a microscopic thoroughness, but not the firm grasp and clear insight which the broader training gives. In fact, if a choice must be made, it is better to shorten, or even to omit the training of the technical school, leaving the man to supply this deficiency for himself, rather than to sacrifice the broader outlook and the wider sympathy which is given by the more diversified training in the outlines of general knowledge.

It is safe to say, therefore, that in the new century he who aims at the highest success as an engineer must be a more learned man than his predecessor of the last century. He must be master of the strictly engineering sciences, and, as a basis and introduction to these, he must have the wider training of the preparatory school and the college.

All this, however, is but the beginning. For, in addition to the training which is to be gained in schools and from books, he must have the training which is to be gained only in the post-graduate school of actual life and work, in which alone the final degree of Engineer, in fact, can ever be attained.

For it must never be forgotten that for real success in any calling, be it that of the professional man or the man in political or commercial life, there is needed something which neither books nor schools alone can teach; something which is partly the result of inherited qualities and partly the result of daily contact with men and things; something which is the resultant of the reaction of our inheritance upon our environment, and which we sum up in the word character. Before there can be real power in any walk in life there must, in addition to technical training, be energy, veracity, self-respect, courage and address. That is to say, before we can have the

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able engineer, or the worthy exemplar of any profession, we must, first of all, and more than all, have a strong and worthy man.

And for the development of these manly qualities what better school can there be than is found in the daily life of the working engineer. For, first of all, in so far as he is an engineer must he be an open-minded and essentially truth-loving man. As the prime condition of his control over the forces of Nature, he must see things and their relative values, not as he wishes them, nor as he already thinks them to be, but exactly as they are in fact. With the facts before him he must have the skill to rightly interpret them, and to this he must add the courage to make his actions conform to his convictions. He must be a man of invention and resource, one who can act promptly as well as see clearly.

At his own peril he must not deceive himself. Otherwise his works will fail and he fail with them. And with equal emphasis is he warned against all attempts to deceive others, for in no calling are such attempts so quickly detected or so certainly futile. A false theory in medicine, in law, or in theology may live, as many have lived, undetected for a thousand years, but a false theory in engineering is disproved by the first attempt to put it in practice. If its embodiment be an engine, it refuses to go; if it be a structure, it falls to pieces. The pretender, the quack, the charlatan, have no abiding place among engineers. They are so quickly unmasked that they speedily disappear. No one, in fact, is so sure to be known at his true value as is the engineer, and no one can so well afford to discard all artificial aids to recognition. What he does is in the sight of all men, and speaks for him.

> "Work of his hand
> He nor commends nor grieves;
> Pleads for itself the fact,
> As unrepenting Nature leaves
> Her every act."

And if it be a question of motive, to whom is the duty and dignity of a life of service brought more closely home than to him whose life-work is found in directing the sources of Power in Nature for the use and convenience of Man. And what greater pleasure and satisfaction can there be than in a life devoted to solving the riddles which Nature everywhere propounds, knowing, as we do, that every new solution means a fresh source of power and benefit to mankind? In fact, the

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whole tendency of the engineer's life and work is to clarify and quicken his intellect, to train his will, and to strengthen every good impulse. How many shining examples and proofs of this are found in the roll of members, living as well as dead, of our own Society, how many have we ourselves known—strong, alert, kindly, clear-headed men—men quick to respond to every call of duty and who could be trusted to the uttermost in every relation of life?

If to a character such as this we add a grasp of the broad outlines of general knowledge and a command of the new sciences which the labors of the last two generations have bequeathed to us, we have the man who will be the typical engineer of the new century. He will be not unlike the engineer of the last century, will, in fact, be his true successor, and, considering his greater opportunities, no more to be commended. Yet, as a man of wider knowledge and ampler intellectual resources, he cannot help being a man of greater power. Great as have been the men, and great as has been the work of the engineers of the last hundred and fifty years, during which the steam engine, the railroad, the telegraph, the telephone, the power press and the myriad other improved tools have revolutionized the conditions of human life, making it, not only longer, but richer and better worth the living, there is reason to believe that even greater work is in store for our successors. Certain it is that, as knowledge grows, the field for its application to the uses of daily life must also grow. Even now, almost every process of manufacture and transportation, as well as all the machinery of war, both on land and on sea, are the work of the engineer. And in the future, even more than in the present, will the secrets of power be in his keeping, and more and more will he be a leader and benefactor of men. That his place in the esteem of his fellows and of the world will keep pace with his growing capacity and widening achievement is as certain as that effect will follow cause.

To aid in this higher education of the American engineer is the special function of this Society, whose objects, as stated in the Constitution, are: "The advancement of engineering knowledge and practice and the maintenance of a high professional standard among its members."

How well it has succeeded in the attainment of these ends the records of the first fifty years in the life of the Society, during which it has become the leading organization of its kind in America, abundantly attest. That, in the years to come, it will be a more potent

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factor in the advancement of knowledge and the promotion of high professional standards, is something which we cannot doubt.

This it will do in part by the intellectual stimulus of its papers and discussions, in which are recorded the latest achievements and the latest ideas of engineers engaged in active work. Even now our *Transactions* are an essential part of the library of every engineering school and of every working engineer. But more important than the intellectual stimulus of our *Transactions* will be the moral stimulus of the personal contact and acquaintance made possible by our meetings. It is in fact hard to over-estimate the value of such reunions in quickening the interest of those who take part in them. Nothing is so vitalizing as the meeting of those of like sympathies and common pursuits. Scattered, the embers may die out; in contact, they burst into flame.

And it is, I think, safe to say that for the kindling of professional enthusiasm, and the establishment of high professional standards, the Society and its members will continue to rely, as they have done in the past, upon these vital and moral forces, and not upon the enactment of codes or upon any form of legislation.

Indeed, anything which savors of compulsion is to be deprecated as not comporting with the dignity of a body of men, chosen, like our own, upon a basis of education and approved character. We may safely trust such men to the guidance of their own sense of fitness and right and to the contagion of noble example.

Moreover, any code that can be framed is certain to be inadequate. It cannot fit every case, and cannot adjust itself to situations that are ever changing.

> "New occasions teach new duties,
> Time makes ancient good uncouth."

And any body of rules, however skilfully framed to-day, will be antiquated to-morrow. As expressions of individual opinion, and suggestions for the guidance of others, such rules may be of great value, but as official codes to compel compliance and take the place of the individual conscience, they will be harmful rather than helpful. But, animated by the inspiration of example, personal contact and friendship, hardly any limit can be set to the effect of our Society in lifting our profession to the highest plane of real service and honors well deserved.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 924.

THE BOHIO DAM.*

By GEORGE S. MORISON, Past-President, Am. Soc. C. E.

WITH DISCUSSION BY MESSRS. FREDERIC P. STEARNS, ALLEN HAZEN, EDWARD P. NORTH, BOYD EHLE, THEODORE PASCHKE, WILLIAM H. BURR, A. G. MENOCAL, H. N. PHARR, EDWIN DUREYEA, JR., C. A. SUNDSTROM, EDWARD WEGMANN, PHILIPP FORCHHEIMER, J. L. CAMPBELL, J. T. FORD AND GEORGE S. MORISON.

All engineers who have examined the route of the Panama Canal agree that the neighborhood of Bohio is the only available location for the dam by which the summit level must be maintained. On the other hand, there have been wide differences of opinion as to how this dam should be constructed, and how high it could safely be built. While it would be an exaggeration to say that the conditions met here are unprecedented, they call for engineering skill and judgment of the highest order. A dam built to meet requirements which have been imposed in other localities would be enormously expensive. The real skill of an engineer is shown in designing the least expensive safe structure adapted to the particular case in hand, even though it may not comply with requirements commonly demanded.

The formation of the Isthmus is generally volcanic, with the irregularities of surface, rocks, and elevations usually found in volcanic countries. The width of the Chagres Valley at any fixed elevation is

* Presented at the meeting of March 5th, 1902.

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very variable, while rocky islands rise here and there above the level of the alluvial plain. The sides of the valley are generally rocky, although much of this rock is of soft character; it has often weathered into a fertile soil, which was formerly covered by a tropical forest, and would quickly return to its pristine condition if the rude cultivation now practiced were abandoned. The bottom land or alluvial plain of the river is comparatively narrow, and is formed of the sediment from a country of much the same character as that which bounds it. As the rocks are of very diverse character the alluvial soil is also variable. It is generally a silt or a compact mixture of sand and clay practically impervious to water, but in some places coarse sand and gravel are found. There are various bars and other obstructions in the river, some of which are rock in position, and others bowlders, gravel or sand.

At Bohio the high banks of the valley are near together, and rock is found quite near the surface on both sides of the river. The river is less than 200 ft. wide at ordinary stages, and a superficial examination would lead one to believe that rock would be found at very moderate depths completely across the valley. The first impression made is that this is an ideal site for a dam, and that a masonry structure could be built at moderate expense. Borings, however, reveal the unwelcome fact that the alluvial deposit has simply filled up an old, deep geological valley, the bottom of which, at places, is 140 ft. below tide level, and that the lower portion of this valley is filled with sand and gravel permeable to water. The course of the geological valley does not everywhere agree with the present bed of the Chagres River, but it lies within the hills which bound the larger valley. Permeable sand and gravel are found in the lowest part of the geological channel, overlaid by a heavy and tight blanket of finer alluvial deposit. There are strong probabilities that this permeable material connects with the river either above or below, or both, the evidence of this being that when a pipe is driven into this material the water in the pipe stands at the same level as that in the river. It is practically established, however, that there is no connection with the river for a considerable distance, probably at least ½ mile, above the dam site. Fig. 1 is a general map of this locality. The immediate location, on a larger scale, is shown in Fig. 2, and several cross-sections of the valley, determined by borings, are shown in Fig. 3.

About 2 miles above the dam site, a local tributary, the Gigante, enters the Chagres from the west. A low summit separates the Gigante

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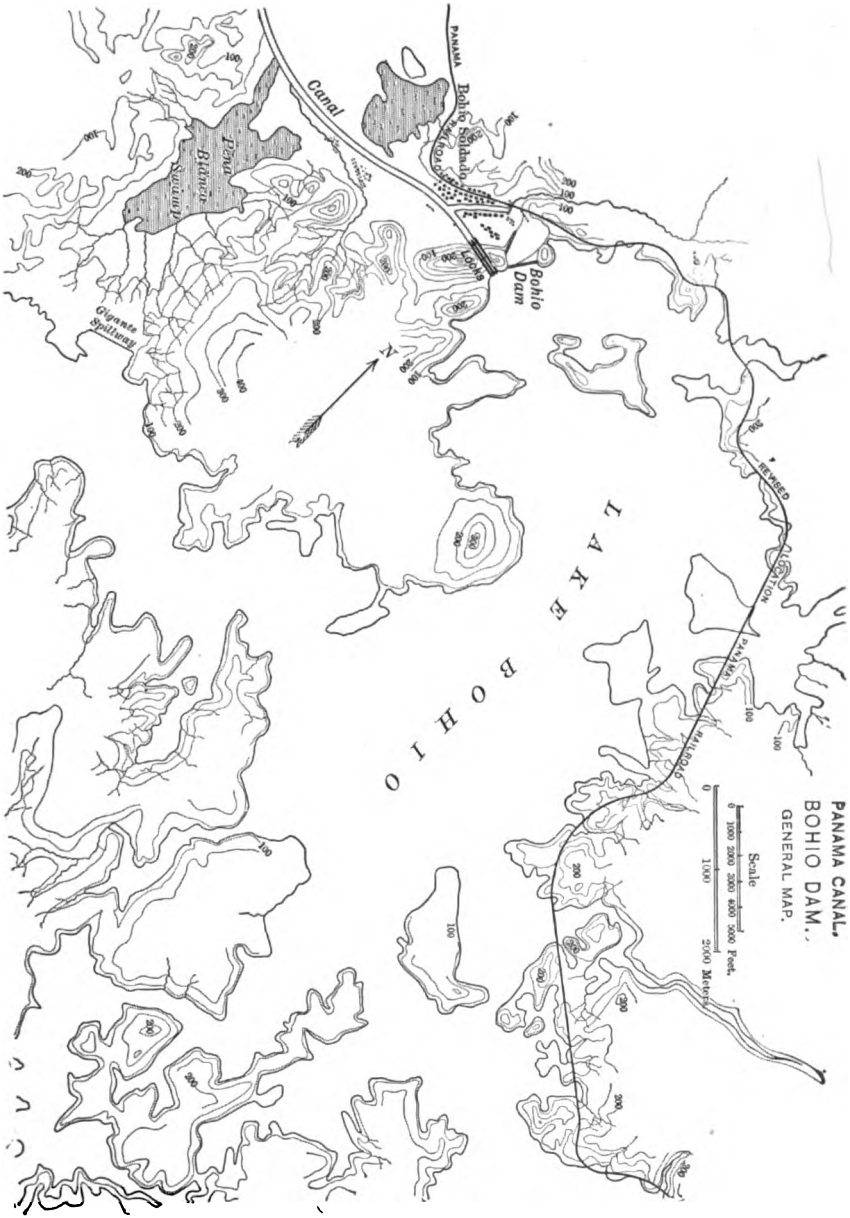

Fig. 1.

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from the Chagres 2 miles below the dam site. The natural summit here is 67 ft. above tide water at the lowest part of the gap. Good rock foundation is found at this place, and it is in every respect an excellent location for a spillway. The floods of the Chagres must either pass through the present course of the Chagres by Bohio, or must pass over the gap at the head of the Gigante. To force them through the Gigante gap requires the construction of a dam, at or near Bohio, at least as high as the gap. Until this is done all the work at the dam site will be subject to interruptions and dangers from floods in the Chagres. There are no special difficulties attending the construction of the Gigante Spillway.

The Gigante Spillway is an ideal arrangement. When the Bohio Dam is completed and the spillway built, the entire discharge of the river will be through the Gigante gap, and the dam will be almost 3 miles away from what will then be the Chagres. This gap, however, is a comparatively late discovery. All the earlier French plans were based on spillways on each side of the Bohio Dam, and not far from it, discharging into diversion channels on each side of and parallel with the canal. The published plans of the New French Company show that its engineers have failed to realize the great advantages of the Gigante location; they provide only for the discharge of a portion of the river by this route, the remainder being taken through a spillway very near the dam. The Gigante Spillway is, however, a superb solution of the problem of the control of the Chagres, the whole work being done by a fixed weir with no complication of gates or sluices. A weir 2 000 ft. long can be built here, which will pass a maximum flood, of any probable duration, with a surface not more than 5 ft. above the crest of the weir, and a maximum flood of indefinite duration, such as has never been known, with a surface barely 7 ft. above. The discharge of the river below can be kept at a safe distance from the canal, and made to pass over a low, marshy country of little value.

The final result of the completion of the works will be that the Chagres will flow through the Gigante gap, where it will pass over a masonry dam founded on rock. Above this dam will be a lake, about 40 sq. miles in area, not unlike many other artificial reservoirs which have been created on rivers. The present valley will be closed by an artificial barrier, this barrier being located about 2 miles from the

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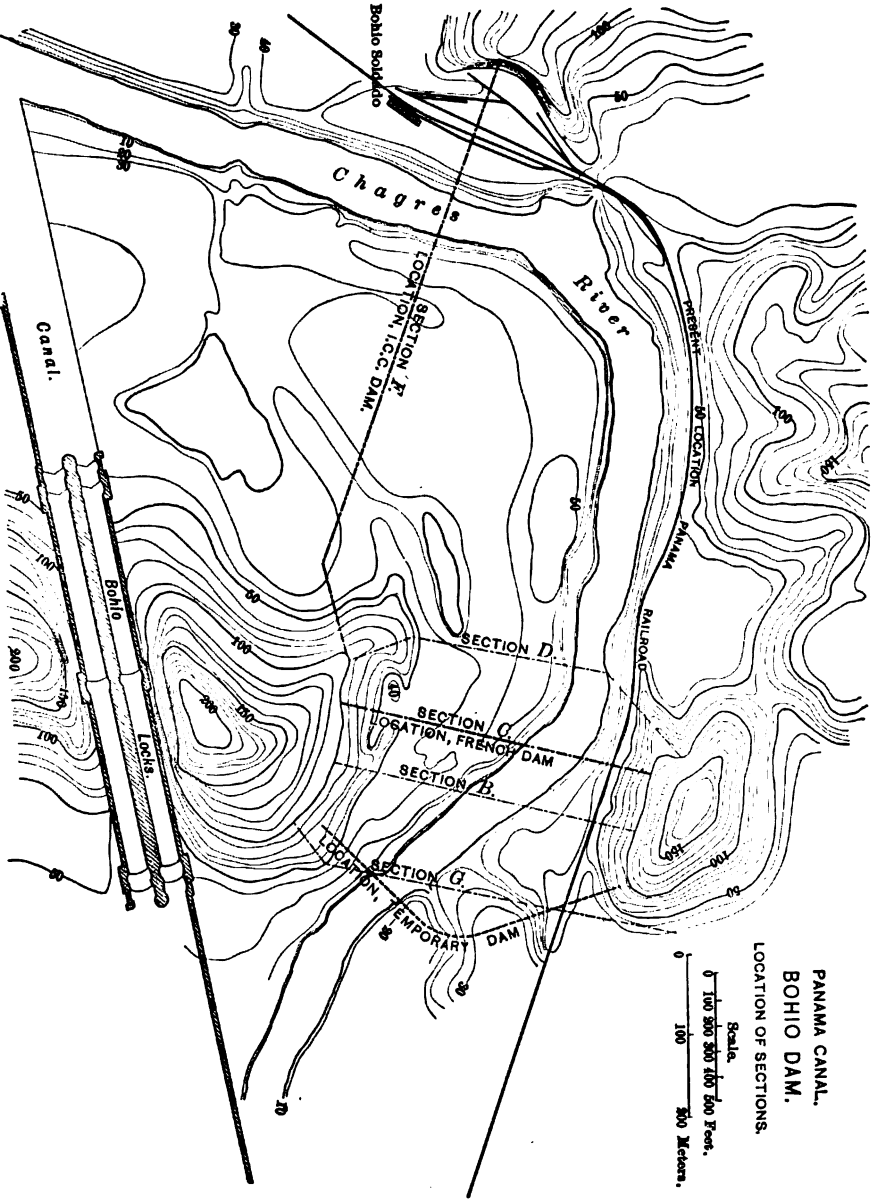
Bohio Soldado
Chagres
River
Canal.
Bohio
Locks.
LOCATION, SECTION F.
LOCATION, C. C. DAM.
PRESENT LOCATION PANAMA RAILROAD
SECTION D.
SECTION C.
LOCATION, FRENCH DAM
SECTION B.
SECTION G.
LOCATION, TEMPORARY DAM
PANAMA CANAL.
BOHIO DAM.
LOCATION OF SECTIONS.
Scale.
0 100 200 300 400 500 Feet.
0 100 200 Meters.

Feb. 2.

course which the Chagres will then take. It will be subject to no possible overflow, nor any currents, but will be simply an artificial connection of the hills which now form the sides of the valley, and which will then form the eastern boundary of the modified valley. The practical result will be the same as when a natural valley is closed by geological disturbances, and a river is forced to seek a new channel through the lowest available gap.

Besides the problem raised by the great depth to rock, the construction of the Bohio Dam is complicated by the necessity of taking care of the Chagres floods while it is building, or at least until they can be diverted through the Gigante gap. While various plans have been suggested for this dam, only three will now be considered. The first of these is the plan proposed by the engineers of the New French Company (Compagnie Nouvelle du Canal de Panama). The second is the plan adopted by the Isthmian Canal Commission, although it was not preferred by the writer. The third is the plan proposed by the writer. These three plans will be considered successively.

The French Plan.—The French engineers located their dam about midway between the two villages of Bohio and Buena Vista, at the place where the valley is narrowest, and directly opposite the site selected for the Bohio locks. This location is shown in Fig. 2. A cross-section of this proposed dam is shown in Fig. 4, and on the same figure is an enlarged cross-section made to correspond to the height adopted for the other dams.

The surface material was to be excavated to a varying depth, but not beyond what it is believed could be done with open coffer-dams; and, starting on the ground so laid bare, an earth dam of compact material, but without a core-wall, was to be built. This dam was to be given a slope of 1½ horizontal to 1 vertical, broken by a reach of 2 horizontal to 1 vertical on the down-stream face, and of 3 horizontal to 1 vertical, broken by four flat benches on the up-stream face; the dam to finish 10 ft. (3 m.) above extreme high water, and to measure 49 ft. (15 m.) wide on top. The entire down-stream slope was to be faced with rip-rap 1 m. thick, this covering to be extended over the greater part of the top. The up-stream surface was to be faced with concrete of equal thickness. A concrete wall was to be built across the river at the foot of the up-stream face, this wall to be built between two lines of sheet-piling. As a protection from possible floods

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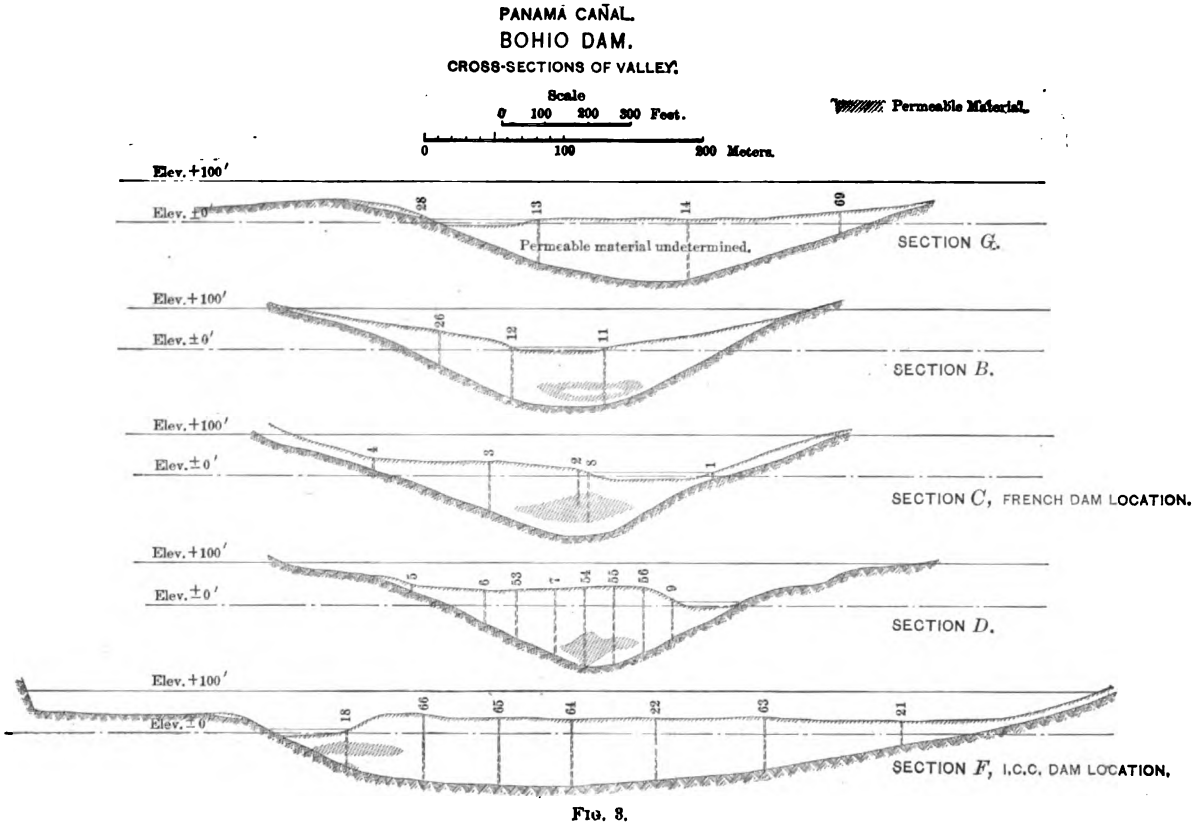

FIG. 3.

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during construction, a large amount of heavy rip-rap was to be placed on the down-stream face, this being designed principally to prevent the destruction of the work in case a flood should pass over the dam before it was completed. The only feature in which this dam can be said to differ materially from many other earth dams is the concrete facing on the up-stream slope, and this must be regarded as of doubtful expediency. A rigid surface of this character could not be expected to adapt itself to the irregularities of settlement, which are always likely to occur in heavy masses of earthwork; and cavities might form, of which there would be little danger if the surface covering were a loose one. This dam, like other earth dams, would not be safe against an overflow, but it would be tight in itself, and the only loss of water would be that which would percolate through the permeable material in the valley below the bottom of the dam. The importance of this seepage will be taken up later.

The Technical Commission, to whom the entire scheme of the New Panama Company was submitted for approval, thought it wise to limit the head of water against this dam to about 66 ft., or 20 m. There seems to have been no reason for this particular limit, except the general judgment, of the engineers forming the Commission, that it was not wise to expose an earth dam to a greater pressure of water.

To facilitate the construction of this dam, it was proposed to build a temporary dam of piles, timber and loose rock about 2 000 ft. above the permanent dam, and, during the construction of the permanent dam, to deflect the passage of the Chagres through the excavation made for the locks, which are but a short distance away. Should an extreme flood occur, exceeding the capacity of this channel through the locks, the work at the dam site would be flooded, and the heavy rip-rap outside the proper limits of the dam on the down-stream face was expected to save the works in case of such overflow. After the dam itself reached an elevation of 14 m. (46 ft.), it was calculated that there would be no further danger of overflow, and this special protection was not to be carried above this level. The construction of the dam could not really begin until the excavation of the locks had been completed, and the work on the locks must, in a great measure, be suspended while the dam is building. It seems to be important that these two works should be made as far as possible independent, which would be accomplished by building a temporary dam of such size that

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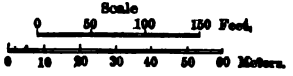
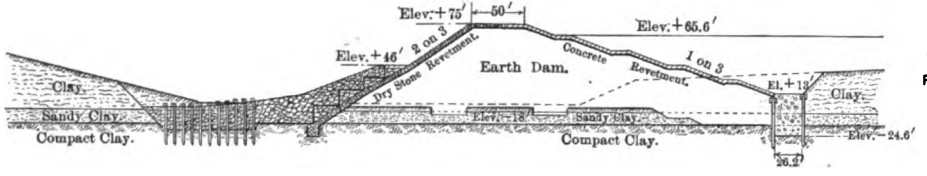
PANAMA CANAL.

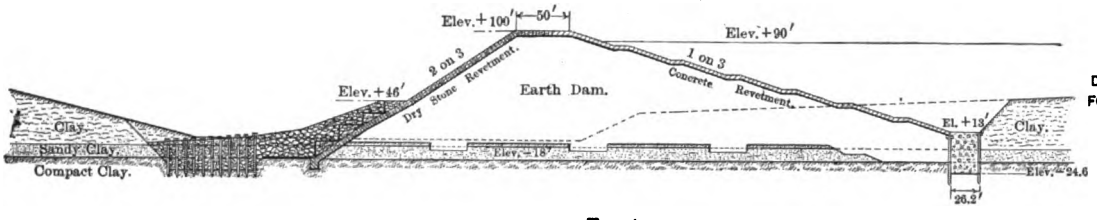
BOHIO DAM.

FRENCH DAM.





DAM AS DESIGNED FOR SUMMIT LEVEL, AT ELEV. 65.6′.



DAM AS ENLARGED FOR SUMMIT LEVEL, AT ELEV. 90.0′.

FIG. 4.

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the diversion would be through the Gigante gap, instead of through the excavation for the locks.

The detailed quantities involved in the construction of this dam, according to the French estimate, and the estimated cost are as given in Table No. 1.

TABLE No. 1.

Earth excavation, above tide level.....	400 000	cu. m.	= 523 200	cu. yds.	at	$0.45	$235 440
" " below " "	62 250	"	= 81 423	"	"	1.00	81 423
Rock excavation.....	20 800	"	= 27 206	"	"	1.15	31 287
Earth placed in dam.....	326 100	"	= 426 539	"	"	0.60	255 923
Concrete and masonry laid in mortar..	29 650	"	= 38 782	"	"	8.00	310 256
Revetment of loose rock.....	18 900	"	= 22 604	sq. yds.	"	2.00	45 208
Loose rock on down-stream face.....	189 800	"	= 182 858	cu. yds.	"	0.50	91 429
Timber in sheet-piling, etc.....	3 260	"	= 1 385	M. B. M.	"	60.00	83 100
Total.....							$1 134 066

The prices in Table No. 1 are those used in the Isthmian Canal Commission's Report, so far as they are applicable, and they are subject to such modifications as the judgment of engineers may think right. This estimate includes no temporary work. This dam is only 75 ft. high above tide water, the other designs being 100 ft. high. If, for purposes of comparison, the estimate be increased in proportion to the squares of these heights, the cost of the higher dam, on the French plan, would be $2 016 030, or, in round numbers, $2 000 000.

The Isthmian Canal Commission's Plan.—The plan adopted by the Isthmian Canal Commission was designed to resist a normal head of water of 90 ft., or about 25 ft. more than the limit arbitrarily adopted for the French dam. The design is based on the theory that the dam must not merely be tight in itself, but must absolutely close the valley, so that there will be no seepage either through it or below it. The plan selected consists of an earth dam 20 ft. wide on top, finishing 10 ft. above high water in the lake, with slopes of 3 horizontal to 1 vertical on both faces, broken by narrow horizontal benches, both surfaces being revetted with loose rock. This dam encloses a masonry core-wall, 8 ft. thick on top and 30 ft. wide at the base, which would be carried down to the underlying bed-rock. The location of this dam is shown in Fig. 2, and the elevation and cross-section in Fig. 5. The location was selected as one on which borings had indicated the maximum depth of rock to be the least, this depth being 129 ft. below tide water. The method of construction would consist in sinking

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PANAMA CANAL.

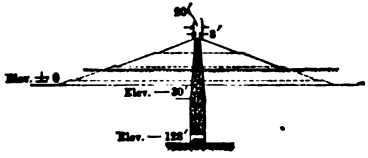
BOHIO DAM.

ISTHMIAN CANAL COMMISSION'S DAM.

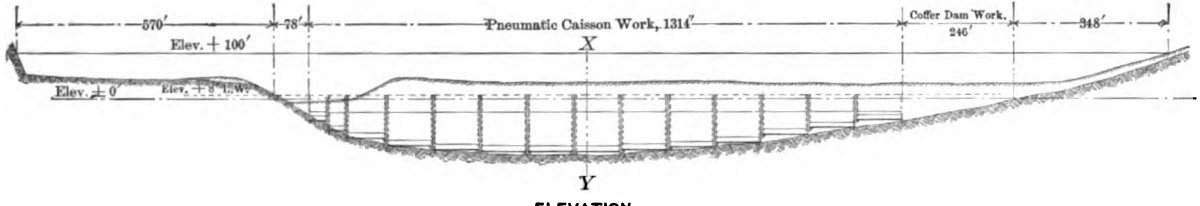
Scale.

0 100 200 300 Feet.

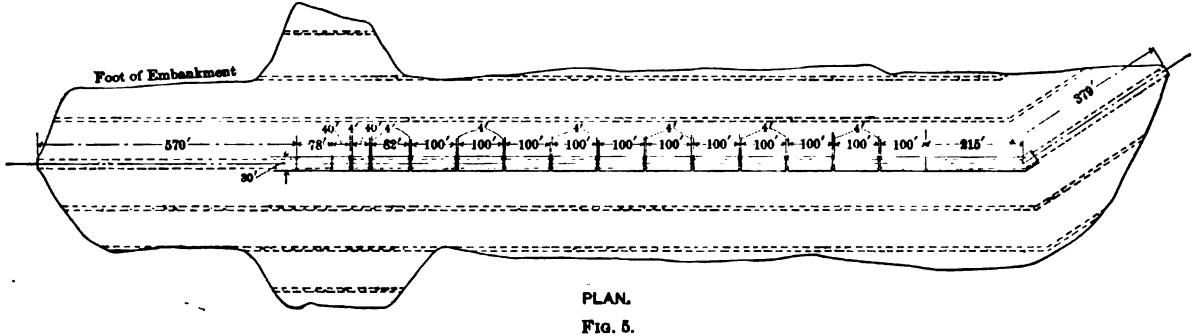
0 100 200 Metres.



SECTION, ON LINE X-Y



ELEVATION.



PLAN.

FIG. 5.

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a series of pneumatic caissons, these caissons to be the foundation of a solid concrete wall, the two together constituting the core-wall. The design involves the extension of pneumatic work to unprecedented depths, and involves special details in making the joints between the caissons tight.

The location selected for this dam is below that chosen by the French engineers, and practically at right angles to the French dam; the length of dam, as well as the cross-section of the valley, on this line being double that on the French location. The desire to limit the depth of pneumatic work led to the selection of this line, but it is possible that additional borings might find a deeper hole here.

There can be little doubt that a dam of this character could be built. It involves novel and untried features. Few engineers, even among those who feel that they could construct it, would be ready to say in advance how the work would be done. The difficulties, taken in connection with the climate and other surroundings, are enormous.

The estimated cost of this dam, taken from the detailed estimates of the Commission, is as given in Table No. 2.

TABLE No. 2.

Earth excavation above tide level	255 500	cu. yds.	at $0.45	$114 975
Excavation below tide level	7 260	"	" 1.00	7 260
Rock excavation	5 730	"	" 1.15	6 590
Pneumatic work	108 410	"	" 21.50	2 330 815
Concrete	286 950	"	" 8.00	2 295 600
Earth placed in dam	1 500 000	"	" 0.60	900 000
Revetment on both sides	107 200	sq. yds.	" 2.00	214 400
Total				$5 869 640

The cost of this dam could be very much diminished by adopting the French location, with its reduced cross-section of valley, and sinking the deeper caissons by dredging through wells, as has been done on many deep bridge foundations, instead of using the pneumatic process. With this method there would be less certainty of a water-tight joint between the caissons and the rock, and between the adjacent caissons. The effect of the core-wall would be to limit the deep-water seepage rather than to close it entirely. In the judgment of the writer, the small difference between the two would not justify the greatly increased cost which the Commission's plan involves.

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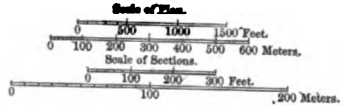
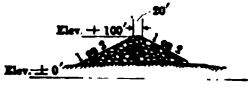
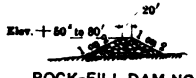
PANAMA CANAL.
BOHIO DAM.
THIRD DAM.

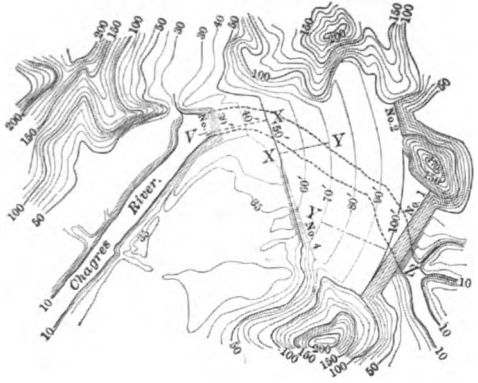


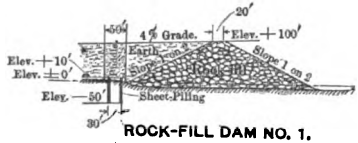


ROCK-FILL DAM NO. 2.



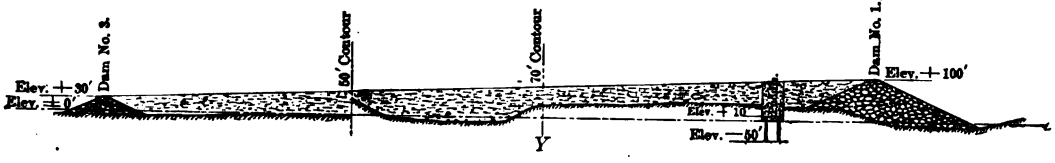
ROCK-FILL DAM NO. 4.





ROCK-FILL DAM NO. 1.

ROCK-FILL DAM NO. 3.



SECTION *VXYZ*.
FIG. 6.

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The Third Plan.—The dam proposed by the writer is designed to sustain a head of 90 ft., or the same as that proposed by the Isthmian Canal Commission. It is based on diverting the Chagres through the Gigante gap during the construction of the dam, and contemplates building a dam of very large dimensions, using material which would otherwise be wasted, this material consisting partly of rock from the excavation for the Bohio Locks and partly of earth excavated from the bed of the canal. The dam is designed to be tight in itself, but no attempt is to be made to prevent seepage through the permeable material below the bottom of the dam, except so far as this can be done by sheet-piling. In this respect this dam agrees with the French plan. Furthermore, after a careful study of the various other locations which the extensive borings made by the Commission indicated might be better, the French location has been chosen as the one best fitted for a structure of this kind, but the dimensions of the two dams differ so greatly that only a portion of the section of this dam really occupies the French location.

The general arrangement of this dam consists of two rock-fill dams, one located about 200 ft. above and the other about 2 000 ft. below the French location. The upper dam would finish at Elevation 100, or 10 ft. above the level of the water impounded in the lake. The lower dam would finish at Elevation 30, or 70 ft. lower than the upper dam. The space between the two rock-fill dams would be filled with impermeable material, the two dams being retaining-walls which would hold this earth in position; the surface of this earth filling would have a grade of 4% sloping from the lake, which is much flatter than any slope of saturation. The upper dam would abut against a hill separated from the high country beyond it by a gap lower than the level of the lake, and a small rock-fill dam of the same description would be required in this gap. Another small rock-fill dam would be required on the down-stream side of the earth filling, at a place where the natural surface of the ground is lower than the proposed filling. All four of these dams would be of the same class of construction, although but one of them would be of any considerable size. For convenience, the upper dam is designated as Dam No. 1, the dam across the gap as Dam No. 2, the low dam across the river as Dam No. 2, and the side dam as Dam No. 4. The location of these dams is shown in Fig. 2, and a cross-section is shown in Fig. 6.

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The four rock-fill dams would all be made on the same plan; they would finish 20 ft. wide on top, with slopes of 2 horizontal to 1 vertical on both sides, and they would be made entirely of loose rock taken from the excavation for the Bohio Locks, which would be deposited here in the same way in which rock fills are ordinarily made. The total amount of excavation for the Bohio Locks is estimated at 1 084 664 cu. yds. The estimated volume of the four dams is 555 000 cu. yds. Allowing for voids in the dams, the excavation for the locks would furnish three times the amount of rock needed for dam construction.

On the line of the French location, and immediately below Dam No. 1, it is proposed to drive two lines of sheet-piles, 30 ft. apart, from one rock bank to the other; these piles to be 60 ft. long, driven 50 ft. below tide water and terminating 10 ft. above tide water; the object being to exclude the flow of water within these limits. They will not check any flow of water at a greater depth, but 50 ft. is considered as great a depth as it is safe to estimate on driving. The upper 10 ft. of these piles would be buried in a puddle wall, 50 ft. thick, which would be laid up as the earth filling proceeds, and would be carefully rolled and compacted as it is made.

The entire space bounded by the rock-fill dams and the hills which they connect would be filled with clay and other earth excavated from the canal, which would be deposited here instead of being wasted elsewhere; it would generally be brought in by trains from the canal below Bohio, but if thought best it could be brought from the great Culebra Cut, the northern spoil bank from which is only 17 miles away. The surface of this earth filling should be planted with some tropical grass, and carefully maintained until all danger of settlement or of surface washing is over, when it might be allowed to grow up into a forest.

The estimated cost of this dam is as given in Table No. 3.

TABLE No. 3.

Rock-fill dam No. 1.....	450 000	cu. yds. at	$0.50	$225 000
" " " 2.....	27 200	" "	0.50	13 600
" " " 3.....	32 000	" "	0.50	16 000
" " " 4.....	46 300	" "	0 50	23 150
Puddle wall.....	150 000	" "	0.60	90 000
Earth filling.....	2 895 000	" "	0.25	723 750
Timber in sheet-piling.....	812 M. B. M.	"	60.00	48 720
Total.....				$1 140 220

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The prices given for rock-fill work (50 cents per cubic yard), and for the earth filling (25 cents per cubic yard), are considered very liberal, as they cover only the additional cost of depositing material here which would otherwise be wasted elsewhere. This is especially true with regard to the rock, the locks being very near the dam. The price for earth is equal to $2 per mile for train service between the Culebra spoil bank and the dam. The same price was used for loose rock deposited on the down-stream face of the French dam. All other prices in all three estimates are the same as those used by the Isthmian Canal Commission in its report.

Temporary Dam.—None of the foregoing estimates includes any provision for a temporary dam by which the waters of the Chagres would be diverted through some other route during the construction of the permanent dam. The French plans contemplated a dam of very small dimensions, and the temporary discharge of the Chagres through the excavation made for the Bohio Locks. Even with the French plan it would be desirable to send the river through the Gigante gap. To do this, a dam must be built which will raise the level of the river above Elevation 65; in other words, it must perform the same duties which the permanent French dam was intended to perform, but the dam may be of more perishable character and a greater seepage may be permitted. In northern climates this dam could be built largely of timber, but the life of timber on the Isthmus is so short that its use must be dispensed with, even for provisional work.

If the Commission's plan is adopted, the best location for the temporary dam would probably be about 600 ft. below the location of the French dam, where the valley is very narrow. If either of the other dams is built, a location must be found further up stream. The position of the temporary dam herein described is shown in Fig. 2.

The location selected crosses the river at right angles about 250 ft. above the foot of the up-stream slope of rock-fill Dam No. 1, and, making an angle near the east bank, connects with the high ground beyond. Rock is found here at reasonable depth for the whole width of the river, though not of the valley. The Chagres is about 150 ft. wide at low water, and the total length of the dam at Elevation 75 is 1 400 ft. The portion of the dam across the river proper must be built

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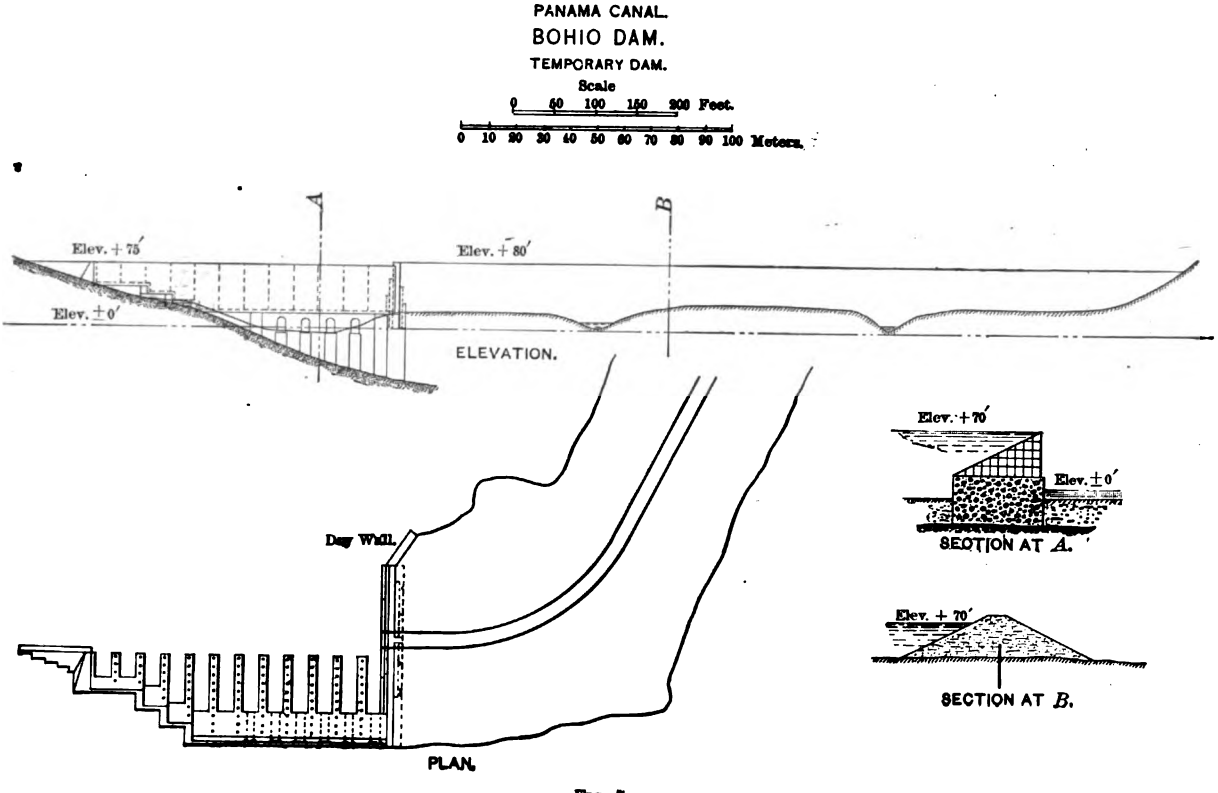

FIG. 7.

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in water; the portion beyond the limits of the river can be built on dry land. The general features of a dam designed for this location are shown in Fig. 7. It consists of two parts, a metal and masonry dam extending across the river, and an earth dam across the bottom land. The foundation consists of a series of separate piers, everywhere founded on rock, the greatest depth being 62 ft. below mean tide and 70 ft. below the water in the Chagres. These piers are connected by arches at the top while the spaces between them are closed by masonry shutters. It is quite probable that all these foundations could be put in with open coffer-dams, but pneumatic work has been estimated on for the four deeper piers. The steel superstructure would consist of a series of bents, spaced 30 ft. apart, and carrying a dam of steel plates. The earth dam is an embankment, 20 ft. wide on top, with side slopes of 2 horizontal to 1 vertical, a loose stone revetment on the up-stream side and a line of sheet-piling driven along the center. The earthwork and masonry could be built at the same time. The openings between the four piers are sufficient to pass the whole discharge of the Chagres during the low-water season. The steel dam could be erected in 90 days; this would be done during the dry season, while the river is passing through the sluices. After its completion, the sluices would be closed by shutters, and the water allowed to rise on the dam. The estimated quantities and cost of this dam are as given in Table No. 4.

TABLE No. 4.

Pneumatic excavation....	22 180 cu. yds., at	$12.00	$266 160	
Open excavation below tide level.....	1 900 " "	1.15	2 185	
Concrete masonry	42 740 " "	8.00	341 920	
Steel.....	1 610 000 lbs., "	0.075	120 750	
Metal dam complete.....				$731 015
Earth embankment.....	200 000 cu. yds. "	0.60	$120 000	
Sheet-piling.....	244 M.B.M. "	60.00	14 640	
Revetment.....	9 900 sq. yds. "	2.00	19 800	
Earth dam complete.....				154 440
Total.....				$885 455

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This seems a very large price for a temporary dam, and the foregoing description must be regarded more as a study than as a definite plan. It is quite probable that a better and more economical structure could be devised. Furthermore, if rock-fill Dam No. 1 could be completed to a safe height above any possible overflow in a single low-water season, the necessity for the temporary dam might be avoided. This rock-fill dam complete is estimated to contain 450 000 cu. yds., and a similar dam finishing at Elevation 75 would contain 300 000 cu. yds. To complete the latter in a single dry season would require an average of 2 000 cu. yds. per day, which would be active work, but not impossible. The site of the main portion of the dam below this rock-fill dam would not thereby be rendered dry, but it would be in a safe condition for work.

Complete Comparison.—If this estimate be added to the previous estimates of the cost of the three respective designs, calling the French estimate $2 000 000 (as all permanent dams are supposed to finish at Elevation 100), the total cost of the three would be as given in Table No. 5.

TABLE No. 5.

	Permanent dam.	Both dams.	With 20% added.
French plan	$2 000 000	$2 885 455	$3 462 546
Commission's plan	5 809 640	6 755 095	8 106 114
Third plan	1 140 220	2 025 675	2 430 810

It is probable that the cost of the temporary dam to be used in connection with the Commission's plan could be somewhat reduced, while there is a possibility that it could be avoided with the third plan, but, in a general way, it may be said that, in the light of our present knowledge, the respective estimates should be, approximately, $3 500 000, $8 000 000, and $2 500 000.

This, however, does not tell the full story. On the American Isthmus the difficulties attending the use of skilled labor are very great. The amount of skilled labor required on the French dam is comparatively small; the Commission's plan requires a large amount; in the third plan the use of skilled labor is practically limited to the construction of the temporary steel dam and its masonry foundation. The third plan really means the construction of an elevated plateau of very large dimensions, the surface area of the earth filling between

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the rock-fill dams being 57 acres. These dimensions may almost be called stupendous. The only dam known of similar dimensions is the so-called North Dike of the Wachusett Reservoir, at Clinton, Mass., the general features of which very closely resemble those of the dam which the writer proposes for Bohio. The criticism which has been made on the third plan is that it permits seepage through the permeable sand and gravel in the lower part of the geological valley.

Seepage.—In considering the effect of seepage, the absolute amount is not so important as the relative amount. The total requirements of the canal have been estimated as 1 000 cu. ft. per second, of which less than 500 cu. ft. would be used for lockage and 200 cu. ft. are required for evaporation, leaving more than 300 cu. ft. for leakage, seepage and other undetermined losses. This quantity (1 000 cu. ft. per second) is equivalent to 28.3 cu. m. per second, which seems very small. On the other hand, it is equivalent to 650 000 000 galls. per day, which exceeds the water supply ever furnished to any city in the world; it is, in fact, an enormous quantity. Except for four months in the year, the discharge of the Chagres very greatly exceeds this amount, and it is possible by additional storage to provide a supply at all times of about 2 500 cu. ft. per second. A loss by seepage, which might prove fatal under a dam sustaining a reservoir for the supply of a fairly large city, would be comparatively harmless in the present case.

Either dam proposed would itself be practically impervious. The puddle wall in the last design may be considered entirely so, and the filling generally would consist of water-tight earth, loose sand and gravel being excluded. The rock-fill dams at the two ends are loose, and would pass water, but the entire structure, including the rock-fill dams, would be founded on water-tight earth, which forms a thick blanket over the sand and gravel below. The distance between the up-stream slope of Dam No. 1 and the down-stream slope of Dam No. 3 is 2 400 ft. in a straight line; the actual distance which water must travel to pass between these points is at least ½ mile. To this should be added the distance from the dam to the place above, where the geological channel may connect with the present river; this distance is not accurately known, but it can hardly be less than ½ mile; in calculating the seepage, however, it has been entirely

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neglected, and the distance which water must travel through a bed of sand and gravel has been assumed to be 2 500 ft.

On the line of the puddle wall, the cross-section of the valley, above the rock and below the line of sheet-piling, is 23 000 sq. ft. At a place 300 ft. farther down the valley the borings showed a cross-section of 10 000 sq. ft. of permeable material. Borings at other points indicate that this is probably a fair average of the amount of permeable material through the entire distance. For purposes of calculation this amount has been doubled, and 20 000 sq. ft. assumed as the average cross-section. The question to determine, then, is the amount of water which will filter through a bed of sand 2 500 ft. long, with a cross-section of 20 000 sq. ft.

Seepage has generally been considered a matter which must be guessed at, rather than calculated, and the opinions of engineers have differed greatly as to what it would probably be. The introduction of filtering plants, however, has led to experiments on the action of different sands in filter beds, and these experiments, while limited in dimensions, have given comparatively definite results, which seem applicable here. In fact, these experiments bear at least as close relations to the case in hand as the hydraulic experiments by which weir formulas have been determined bear to a dam across a great river. So far as there are any indications, it would appear to be probable that these filter-bed formulas, when applied to sand beds of very great thickness, will give results too high, rather than too low. The formula deduced from experiments* made by Allen Hazen, M. Am. Soc. C. E., is

$$V = c\,d^2\,\frac{h}{l}\,\frac{t\,\text{Fahr.} + 10}{60}$$

in which V is the speed, in meters per day, at which the water will issue from the entire cross-section of sand; c is a constant, varying from 450 to 1 200, according to the cleanness of the sand; d is the effective diameter of the sand, in millimeters; h is the head; l is the distance which the water must pass through the sand, and t Fahr. the temperature of the water, in degrees Fahrenheit. The effective diameter is the diameter of a sphere the volume of which is greater than that of the grains which form one-tenth of the weight, and less than that of the grains which form nine-tenths of the weight of the whole mass.

* Report of Massachusetts State Board of Health, 1892, page 553.

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Putting $c = 1\,000$; $h = 90$; $l = 2\,500$, and t Fahr. $= 80$, the formula becomes

$$V = d^2 \frac{1\,000 \times 90 \times 90}{2\,500 \times 60} = 54\, d^2.$$

If V_s represents the speed in feet per second, we have

$$V_s = 0.000039\ V = 0.002\, d^2.$$

The value of d is determined by an inspection of the sand and gravel. Eight different specimens of sand and gravel taken from bore holes were analyzed by Mr. Allen Hazen, and the effective diameters reported as follows: 1.3, 2, 0.9, 0.22, 1.6, 0.38, 0.49, 0.57. The average of these is 0.93, but, if they are averaged by their squares, the average is 1.11. As these were selected as specially coarse specimens, they undoubtedly give a much higher value than that of the average material in which the flow in the pipes was lost. Furthermore, they are all washed specimens brought up from the bottom, and do not fairly represent the actual condition of the material in position. If the value of d be assumed as 1 mm., which is only 10% less than the average by squares, the value is probably at least double that which actually exists. Putting $d = 1$, we have

$$V_s = 0.002.$$

Applying this value of V_s to the assumed cross-section of permeable material (20 000), the total seepage would be 40 cu. ft. per second, an insignificant amount compared with the total water consumption of the canal. It is only about one-fifth that required for evaporation. Furthermore, it is probable that this amount is grossly over-estimated. If the value of d be taken at 0.5, the value of c at 500, and the value of l at 5 000, this seepage will be reduced to $2\frac{1}{2}$ cu. ft. per second. Even if, at the beginning, the conditions agree with this assumption, the sediment in the flood water of a few years, will change the value of the several constants to those last named. As these conclusions may be considered remarkable, it is thought worth the while to call attention again to the large figures with which we are dealing. The quantity $\frac{1}{2}$ cu. ft. per second is about 1 600 000 galls. per day, an adequate water supply for a manufacturing town of 40 000 people, or for the whole population now living on the Isthmus.

The speed at which 40 cu. ft. would issue from a surface equal to the cross-section of the permeable material is 0.002 ft. per second. The rate at which it travels through the voids in the sand, however, is very much greater, but would not exceed 0.008 ft. per second, or $\frac{1}{4}$ in.

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PANAMA CANAL.

BOHIO DAM.

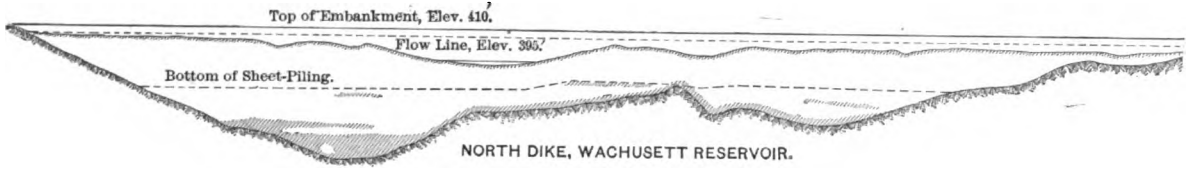
COMPARISON WITH WACHUSETT RESERVOIR.

Scale.

0 100 200 300 Feet.

0 50 100 200 Meters.

Permeable Material.



SECTION *C*, BOHIO DAM.

FIG. 8.

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per minute, being about the speed of the minute hand of a fair-sized clock. While a large aggregate of water can pass through a large section of sand at this low speed, the speed is not enough to move any material except on an open exposed face of comparatively steep slope. As the outlet, if any exists, is considerably below tide level, such disturbance is impossible.

Wachusett Reservoir.—Reference has already been made to the North Dike of the Wachusett Reservoir, near Clinton, Mass., where the situation bears a striking resemblance to that at Bohio. In both instances a large reservoir or lake is created by a dam across a river, and in both instances this dam must be supplemented by an embankment or another dam across a gap lower than the level of the lake. At the Wachusett Reservoir, the masonry dam closes the present channel of the river, and the dike or embankment is across a low gap which marks the site of the geological valley. On the Isthmus, the masonry dam is across the low gap, and becomes the spillway, while the embankment is across the present river, but not across the river as it will be when the works are completed. At the Wachusett Reservoir, the geological valley was filled with drift, and the deflection to the site of the masonry spillway was made by glacial action. On the Isthmus, the deflection from the geological valley to the site of the spillway must be made by artificial means, no glaciers ever having reached that tropical country. When the work is done, the two conditions will be substantially the same.

The head of water on the two sides of the dams will not differ in the two locations enough to affect the percolation materially, the water in the Wachusett Reservoir being more than 100 ft. above the water in the river below, although the actual height of the embankment on its immediate location is somewhat less. The depth of the geological valley is much greater at Clinton than at Bohio, as is also the cross-section of the permeable material. For the purpose of comparison, the two cross-sections have been drawn on the same scale in Fig. 8.

The dams differ principally in the character of the material used for their construction. In place of rock-fill dams, gravel embankments are being used at Clinton; and, in place of material taken from the canal, the strippings of the lake, or surface soil with a considerable amount of vegetable mould, have been used for filling between these embankments. The same principles govern the percolation and seepage in both cases.

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DISCUSSION.

FREDERIC P. STEARNS, M. Am. Soc. C. E.—Mr. Morison, in his interesting comparison of proposed designs for the Bohio Dam, has called attention to the resemblance between the design prepared by him and the North Dike of the Wachusett Reservoir; and the speaker, thinking that it might be of interest to the Society to have at this time some further information with regard to this structure which is now being built, has prepared the following description of its design and construction. Mr. Stearns.

The preliminary designs of the dike were made by the speaker in 1894, in connection with the preliminary investigations for the Metropolitan Water Supply made by the Massachusetts State Board of Health. The design is an unusual one, and was the result of unusual conditions.

The Nashua River, at the site of the main dam, flows through a narrow gorge between rocky hills covered with a comparatively thin layer of drift material, and these hills rise far above the proposed water level of the reservoir. Between the hill on the north, however, and another hill nearly two miles to the west, there is a sandy plain having a general level about 15 ft. below the water level in the reservoir. In some portions of the plain small rocky hills or ridges rise above its surface; in other portions of the plain, there are depressions and gullies.

North of Sandy Pond, which lies in a very deep depression, there is a deep cut through the plain, through which a small stream from this pond formerly flowed, but the building of a mill dam on a stream north of the plain backed the water up into Sandy Pond, and, before the construction of the dike, the water stood in the cut to a depth of from 5 to 12 ft., for a width of about 300 ft. The mill pond formed by the dam, including the arm connecting it with Sandy Pond, was known as Coachlace Pond. The bottom of this pond, at the point where the dike crosses it, is 65 ft. below the proposed water level. Upon making borings, it was found that the rock was far below the surface of the plain, in one place as much as 286 ft., so that it would be impracticable to reach it with any core-wall.

The geological condition of the drift material over the rock, however, was much more satisfactory, as it was found quite generally that the material above the rock, with the exception of the upper layers, was a very fine and nearly impervious sand.

It had been decided, upon sanitary grounds, to strip the surface soil from the whole of the 6½ sq. miles comprised within the limits of the reservoir. The great depth of the reservoir, and the conformation of the sides, made it nearly as cheap to deposit this material at the site of the dike as elsewhere. The abundance of material available at

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Mr. Stearns. a nominal cost, the fact that less water would percolate through a dike of great thickness than through a thin one, and the desire to provide absolute safety where such an enormous amount of water was to be stored, led to the adoption of a design which is characterized by very unusual dimensions.

Dimensions of the Dike.—The dike covers an area of 143 acres, and will contain 5 500 000 cu. yds. of material. It is 2 miles long on the water side at full-reservoir level, 65 ft. high at the deepest place, up to this level, and has a maximum width of base of 1 930 ft.

In order to prevent the percolation of water through the upper layers of the plain, which, in most cases, consisted of coarse sand or fine gravel, an excavation known as the cut-off trench was made. This has a total length of 9 556 ft., a bottom width of 30 ft., and a maximum depth of 60 ft., and extends longitudinally under the crest of the dike. For 3 124 ft. the trench was excavated to the solid rock, and for the remaining 6 432 ft. into the fine sand.

Wherever the borings indicated that the material at a level below the cut-off trench was somewhat pervious, or where they showed streaks of the coarser sand, sheet-piling was driven in the bottom of the cut-off trench. This sheeting was driven for a total length of trench of 5 245 ft., leaving 1 187 ft. of trench without sheeting.

The portion of the dike easterly from a central rocky ridge, known as the Easterly Portion of the dike, presents the most interesting problems, and is shown in plan by Fig. 9, and in cross-section by Fig. 10. The characteristic features of the design are best shown on the cross-sections, one of them through the highest part of the dike, and the others through the higher ground where the water will have a depth against the dike of only 13 and 20 ft., respectively, but where the pervious character of the upper layers made it necessary to excavate a deep cut-off trench and fill it with impervious material.

The top of the dike is to be 17 ft. or more above the full-reservoir level, with a view to having it not less than 15 ft. above this level in the future after the material has settled.

The distance through the dike at the full-reservoir level is 189 ft. The down-stream slope, as a rule, is only 3%, but there are exceptions to this rule, and in some places the slope is as great as 6 per cent.

The water face of the dam has a slope of 2 horizontal to 1 vertical, both above and below a berm 15 ft. in width and 13 ft. below the full-reservoir level.

Materials Used in the Dike.—As already indicated, soil stripped from the reservoir is used for by far the greater part of the embankment of the dike. This was found, by experiments which will be described subsequently, to be practically impervious, and, except for a tendency to settle under pressure, to have a high degree of stability, and was, therefore, used for filling the cut-off trench as well as other parts of the embankment.

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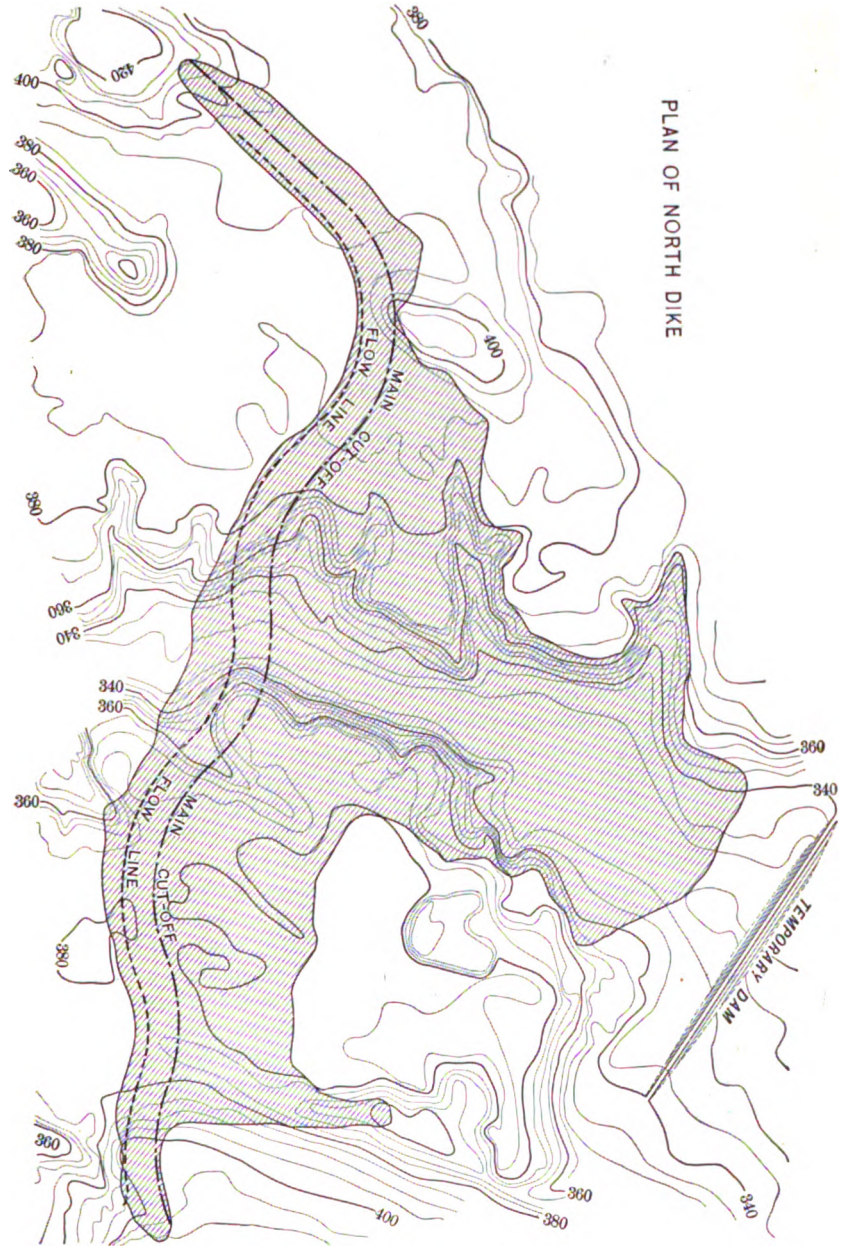

FIG. 9.

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Mr. Stearns.

The water face of the embankment is to be protected above the berm with a layer of coarse gravel, 7 ft. in thickness, resting on an embankment of either sand or gravel and covered with a well-laid paving, 3 ft. in thickness, of large stones embedded in and chinked with broken stone. The paving will extend from 8 ft. above the water level down to the berm, but will diminish in thickness toward its upper and lower ends. Below the berm, light rip-rap may be used in exposed places, but, as a rule, a layer of coarse gravel will be sufficient.

It seemed advisable to give the cut-off trench a considerable width, and it was found to be both better and cheaper to excavate a trench with slopes rather than with vertical sides protected by sheeting. The open excavation, besides being cheaper to make, facilitated the work of driving sheet-piling. It furnished most of the material for the sand and gravel embankment at the water face of the dam, and its form is much more satisfactory than with vertical sides, where the trench is to be filled with a material which will compress as it is loaded more and more by the filling of material above it.

A secondary cut-off, of somewhat smaller dimensions, was provided for a part of the length of the dike, in the position shown on the cross-sections, as an additional precaution which could be provided at a small expense. A secondary embankment of sand and gravel was also added, to meet a suggestion that the soil might lack stability.

The sheet-piling in the bottom of the cut-off trench was an important part of the work, which was accomplished in a very satisfactory manner. The borings showed the presence of fine sands, not wholly impervious, to a depth of 50 ft., in many places, below the bottom of the trench, which made it desirable to use unusually long sheeting if a method of driving could be devised which could give satisfactory results. The entire absence of stones or other coarse material indicated that good results might be obtained with a water-jet.

As it was impracticable to obtain single planks for sheeting of such great length, it was decided, where the lengths exceeded 30 ft., to make the sheeting 6 ins. in thickness and to build it of 2-in. planks in substantially the same manner as the Wakefield triple-lap sheet-piling. Where the length was less than 30 ft., 4-in. grooved spruce sheeting was used. The 2-in. planks were of standard widths, planed to an even thickness and nailed together, except at the lower end, where they were bolted to resist water pressure. The piles were beveled at the bottom for a part of their width, so that they would hug the pile against which they were driven.

Spruce lumber, costing about $18 per 1 000 ft., B. M., was used for all except a short length at the bottom of the pile, where oak was substituted.

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Mr. Stearns.

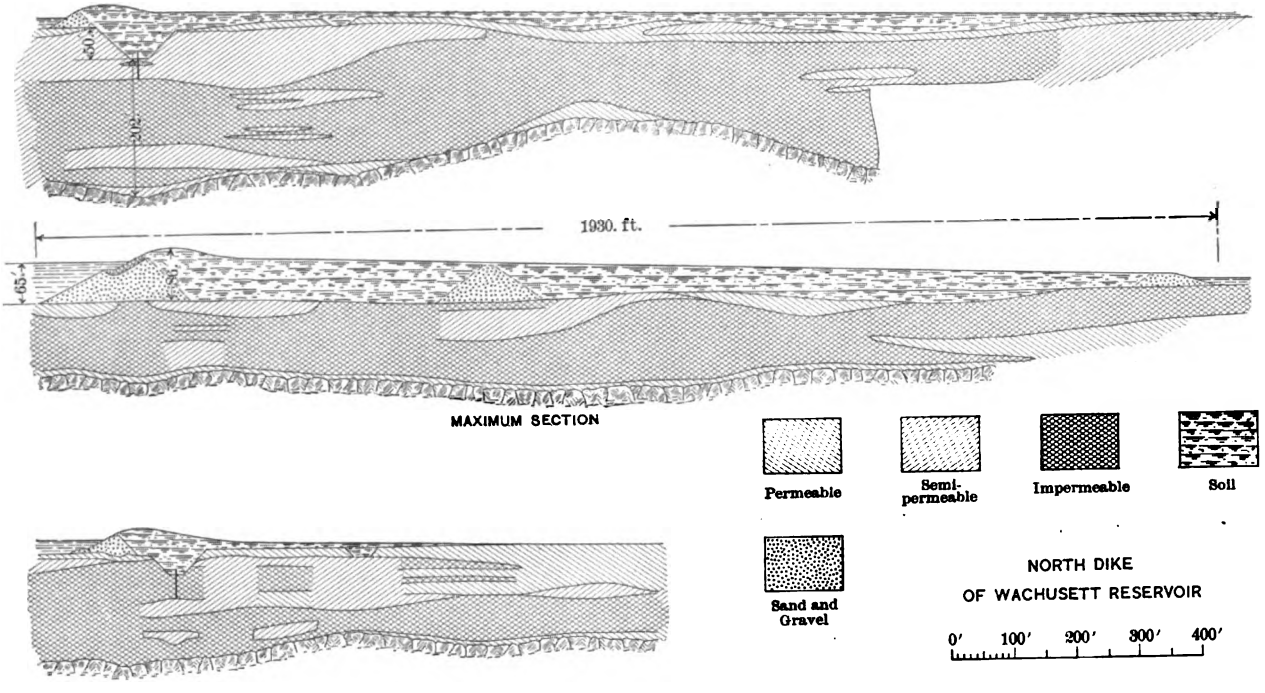

FIG. 10.

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Mr. Stearns. The piles were made of varying widths, generally between 16 ins. and 2 ft., and they were quite commonly from 45 to 50 ft. in length. The longest pile used was 67 ft., and as they were left with their tops 5 ft. above the bottom of the trench in order to make a suitable connection with the soil filling, the greatest depth that a pile was driven into the ground was 62 ft.

For driving the long piles, a pile-driver, 50 ft. high under the hammer, was used, and a water-jet was used in all cases. A large steam pump was installed at the bottom of the trench, and a 6-in. wrought-iron pipe extended nearly to the pile-driver. From this the water was conveyed to the pile by a 3½ or 2½-in. hose, or by a combination of the two, fastened by a detachable coupling to an opening which extended from the side of the pile, near its bottom, to a nozzle at the bottom of the pile. Various kinds of nozzles were used, to meet different circumstances. Each of them furnished a heavy stream pointing directly down, and in general there was a stream spurting back toward the pile already driven, to wash away all sand from between the two piles. The large quantity of water used came to the surface around the pile and also around the hose, keeping them free from pressure. The hammer of the pile-driver was used merely to overcome the friction of the pile against the wedges and the pile previously driven, and to overcome its buoyancy after the pile was driven to a considerable depth into the ground. When the pile had been driven to the full depth, a key, fastened to a small wire rope, was pulled, releasing the hose, which could then be drawn to the surface.

It had been thought, before beginning the work, that it might be desirable to drive two rows of sheeting, but the character of the work done was so satisfactory that it was concluded that one row would be sufficient. The largest number of piles driven at one place in any one day was 28. These piles were 20½ ins. wide and between 45 and 46 ft. in length. They were driven into the ground from 40 to 41 ft.

The method of constructing and driving the sheet-piling was devised by, and the work executed under the direction of, Hiram A. Miller, M. Am. Soc. C. E., Engineer of the Reservoir Department.

Where the trench was excavated to the rock, the surface of the rock was carefully washed off with a hose, using water under a considerable pressure. All seams were carefully cleaned out and, if small, filled with Portland cement mortar, or if large, with bricks laid in the mortar. Brick cut-off walls were built where needed, and the entire rock surface was covered with two coats of grout put on with whitewash brushes. It was the purpose, in all this treatment of the rock, to prevent any possibility of contact between a concentrated stream, which might possibly follow a seam or narrow channel in the

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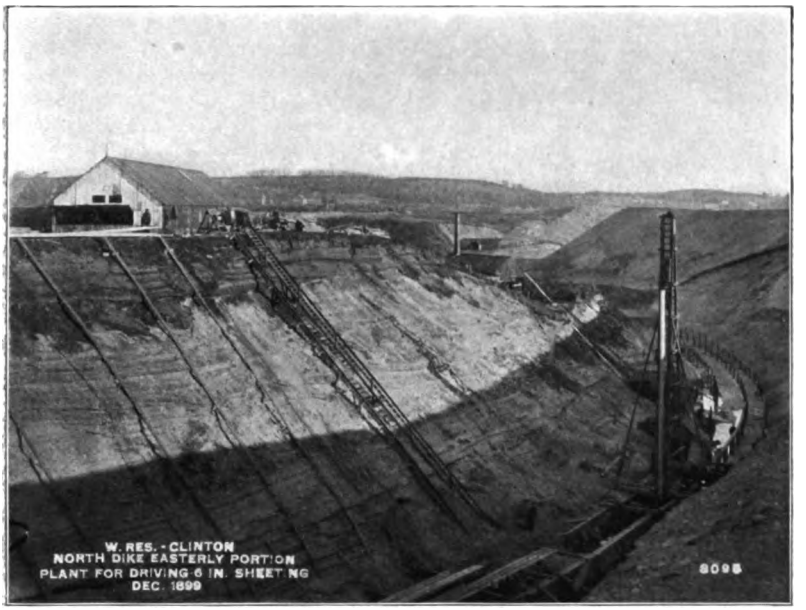

FIG. 1.—NORTH DIKE: CUT-OFF TRENCH AND PLANT FOR DRIVING 6-INCH SHEETING.

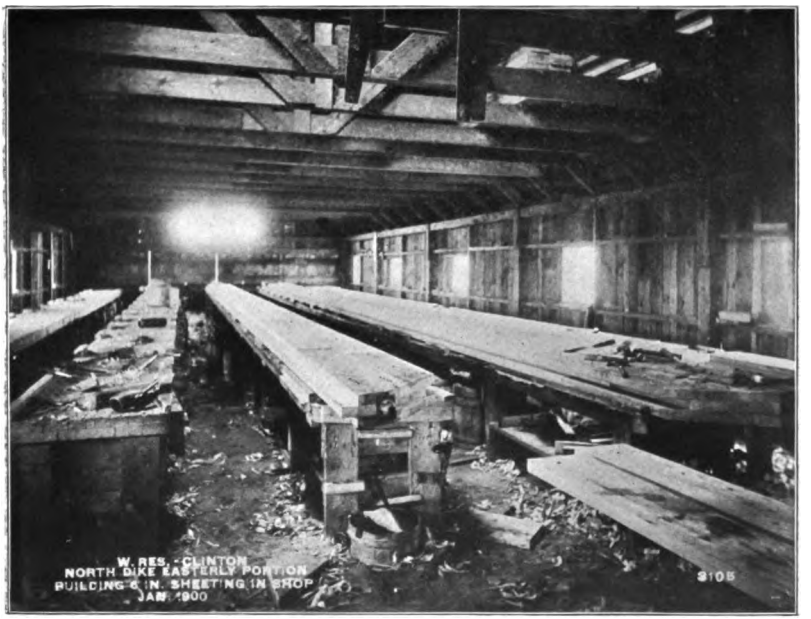

FIG. 2.—NORTH DIKE: BUILDING 6-INCH SHEETING IN SHOP.

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rock, and the soil resting on the rock. The area of the rock surface treated in this way was 77 250 sq. ft. Mr. Stearns.

Plates X to XII are views of the cut-off trench and sheet-piling in various stages.

Fig. 1, Plate X, and Fig. 1, Plate XI, are views of the cut-off trench where sheet-piling is being driven, Fig. 1, Plate XI, showing the 6-in. pipe leading from the pump nearly to the pile-driver, and Fig. 1, Plate X, showing on the left the building in which the piles were built, and the means used for sliding them down to the bottom of the trench.

Fig. 2, Plate X, shows the interior of the building where the piles were built.

Fig. 2, Plate XI, shows the lower portion of a pile just before connecting the hose.

Fig. 1, Plate XII, shows the method of treating the ledge at the bottom of the cut-off trench.

Fig. 2, Plate XII, shows a portion of the cut-off trench nearly filled with soil rolled in 6-in. layers.

At the extreme down-stream toe of the dam, an embankment of very coarse gravel was formed. Its maximum section is 19 ft. high and 115 ft. wide at the base. This embankment is for the purpose of permitting any water which may find its way through the dike to escape without the sloughing of the material. If the quantity filtering should prove to be large, there is a coarse gravel stratum below the surface which could be tapped by means of driven wells to furnish an escape for water filtering through that stratum, but it is not anticipated that these wells will be needed.

Cost of the Dike.—The amount expended on the dike up to the present time is about $500 000, and about two-thirds of the work has been completed.

In looking at the cross-sections, one can hardly realize the relative cost of the different materials. The quantities and costs of the completed dike will be approximately as follows:

Soil	5 250 000	cu. yds.,	at $0.05	$262 500
Cut-off trench	542 000	"	" 0.20	108 400
Borrowed earth and gravel	200 000	"	" 0.20	40 000
Slope-paving	50 000	"	" 2.20	110 000
Sheet-piling, pumping, consolidating soil with water, and other day's labor work				117 000
				$637 900
Engineering and preliminary investigations				120 000
				$757 900

The cost of the soil filling cannot, of course, be very definitely determined, as it is an estimated part of the cost of stripping and dispos-

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Mr. Stearns. ing of the soil from the reservoir, of which the total cost is from 20 to 39 cents per cubic yard. Of the total cost of construction, exclusive of engineering and preliminary investigations, amounting to $637 900, only about two-fifths is chargeable to the great mass of soil of which the dike is composed, and only one-sixth to the cut-off trench, which is of such unusually large dimensions. On the other hand, the slope-paving and sheet-piling, which occupy very little space upon the cross-sections, together represent nearly one-third of the total cost. The cost per linear foot of dike, including the engineering and preliminary investigations, will be about $75.

The Bohio Dam, as designed by Mr. Morison, has two-thirds the bulk of the North Dike, and his estimate of cost, exclusive of protective works, engineering and contingencies, is one-half greater than the cost of the North Dike.

Investigations on which the Design was Based.—In making the original design of the dike, the writer was much indebted, for information with regard to the filtration of water through fine materials, to the investigations of Allen Hazen, M. Am. Soc. C. E., upon the physical properties of sands and gravels.* As indicated by Mr. Morison, Mr. Hazen determined the laws of the flow of water through sands and gravels with such accuracy that he was able to make a formula to express the results. By the application of this formula, it was evident that the very fine sands found at a considerable depth below the surface would not permit enough water to pass through them, if a dike of great width were constructed, to cause a serious loss of water, and it was also found that the soil, which contains not only the fine particles of organic matter but also a very considerable amount of finely comminuted particles, which the geologist has termed "rock flour," would be sufficiently impermeable to be used as a substitute for a clay puddle. The final design, however, was not adopted until after a most extensive series of borings had been made to determine the nature of the material at all depths in all parts of the plain, and an independent series of investigations had also been made with the materials to be found at and near the site of the dike, to determine whether they would be suitable for use.

Borings.—The plain at the site of the dike was so wide that it permitted a large range in the choice of locations, so far as could be judged from surface indications, and it was necessary, therefore, to locate the dike by means of borings.

Owing to the great length and breadth of the territory to be explored and the great depth to the rock, this investigation was a most tedious and expensive one.

Borings were begun with three gangs on April 1st, 1896; three more gangs were added on August 15th of the same year, and the work was

* Report of Massachusetts State Board of Health for 1892, page 541.

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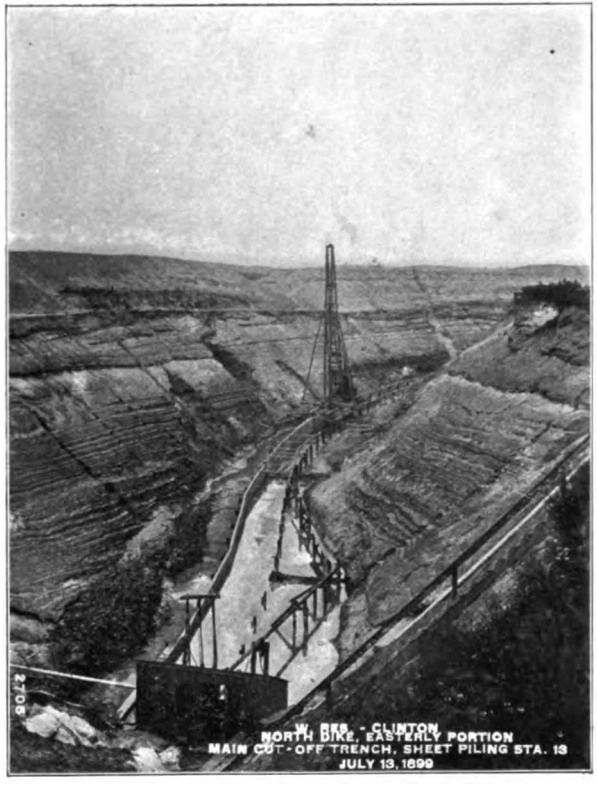

FIG. 1.—NORTH DIKE: MAIN CUT-OFF TRENCH.

FIG. 2.—CONNECTION OF HOSE WITH PILE.

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then continued without interruption, nearly all of the time with six gangs, until December 13th, 1897. A few additional borings were made in 1898. In all, 1 131 borings were made, having an aggregate depth of 93 553 ft., equal to about 17¾ miles. The average depth was 83 ft., and the maximum depth 286 ft. Mr. Stearns.

The borings were the ordinary wash-drill borings, made by driving a 2½-in. pipe into the ground and washing the material encountered to the top of the pipe by a stream of water forced into the bottom of this pipe through a 1-in. wash pipe armed with a drill at its lower end.

In many instances the outer pipe was driven only through the layers of coarser material, and the 1-in. pipe was forced down into the fine sand, in which a hole could be maintained without a casing. Special methods were used to obtain trustworthy samples of the materials encountered.

Professor W. O. Crosby, Geologist of the Massachusetts Institute of Technology, co-operated with the engineers in regard to the manner of conducting the borings, collecting the samples and interpreting the results.

The samples were classified largely by their appearance, but other features were also considered. When the water coming up the pipe while making the borings was discolored by clay, the fact was noted in the records, and in such cases a sample would be classified as more nearly impervious than if this note had not been made. The presence of clay in the samples could also be determined by chemical tests, even though a large part of the clay had been washed away and lost. Many of the samples were used for filtration tests, upon a small scale, and these and other tests affected the classification.

Most of the borings at the easterly half of the dike were made where the ground-water level was from 35 to 50 ft. below the surface, and it is an indication of the comparative impermeability of the material that the water discharged by the 1-in. pipe generally came to the surface in a strong stream, notwithstanding there was a head of from 35 to 50 ft. tending to force the water into the ground, and that in many cases the sides of the very deep holes were not cased and the water could percolate into them as freely as the material would permit. There were places, however, where the stream of water flowing through the 1-in. pipe was wholly absorbed by the ground, and such places were noted in the records and served as an indication of the degree of permeability of the strata in which the water filtered away.

Experimental Dike.—For the purpose of making experiments with a dike built of soil, and for other experiments, a building, 70 ft. long and 25 ft. wide, was constructed. Inside this building a water-tight wooden tank, 60 ft. long, 6 ft. wide and 8 ft. high, was built of 2-in. tongued and grooved pine planks, planed on both sides. Within this

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Mr. Stearns. tank a dike of soil was constructed, as shown in Fig. 11, 8 ft. high, 7 ft. wide on top and with a slope of 2 horizontal to 1 vertical on the up-stream side and 4 horizontal to 1 vertical on the down-stream side, except at the extreme lower end of the slope on the down-stream side, where there was a box, 18 ins. high, filled with gravel, and having perforated sides, so as to allow the water to filter out at the toe of the slope without carrying the soil with it. Immediately over the top of the dike, for a width of 7 ft., there was placed 3 ft. of soil, so as to slightly consolidate the top of the dike and permit the water to be filled up level with it without overflowing.

After the dike had been formed, the upper end of the tank was filled with water to a depth of nearly 8 ft. In order to determine the pressure in different parts of the dike, horizontal pipes were laid across the tank through the soil, near its bottom, at intervals of from 6 to 12 ft. These horizontal pipes were perforated and covered with wire gauze, and at the end of each there was a vertical glass tube. As soon as the soil surrounding the pipes became saturated and under pressure, the amount of pressure would be shown by the height of the water in the glass tubes.

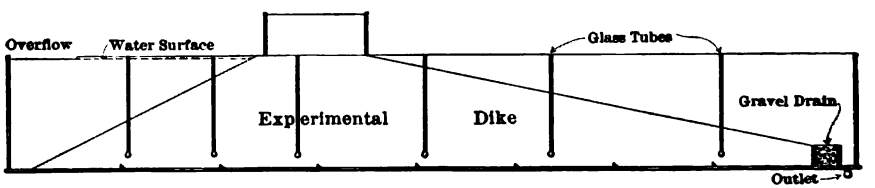

FIG. 11.

Experiments were made on dikes with the soil deposited in different ways:

1. When shoveled loosely into the tank without consolidation of any kind.
2. When deposited by shoveling it into water.

The soil, which had been thrown in loosely, settled as it became saturated by the water and became quite compact. After the pressure had been on for several weeks, the filtration through the dike amounted to only about 1 gall. in 22 minutes. It is interesting to note that it was five days after the water pressure was put on the dike, before any water appeared in the sixth gauge pipe near the lower end of the dike.

In the second experiment the filtration amounted to 1 gall. in 34 minutes.

In a third experiment, made under nearly the same conditions as the second, the rate of filtration was 1 gall. in 35 minutes.

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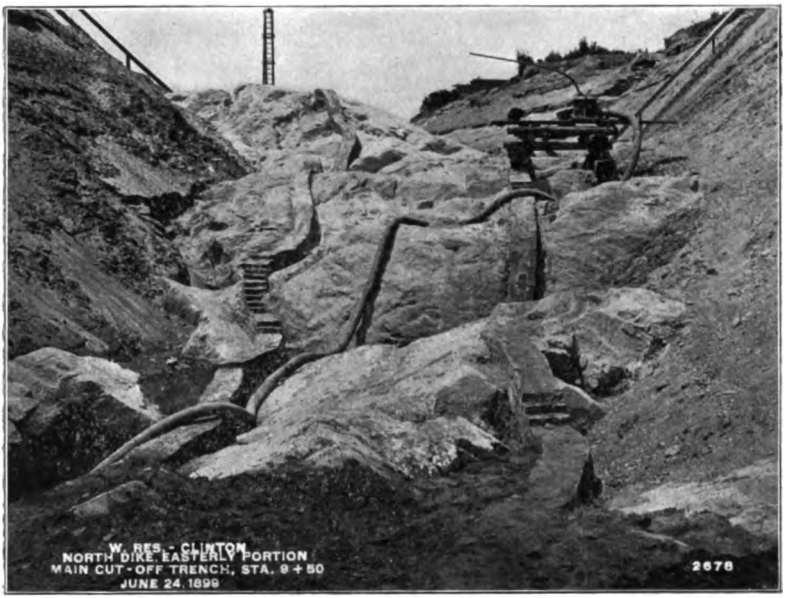

FIG. 1.—NORTH DIKE: CUT-OFF TRENCH, SHOWING TREATMENT OF LEDGE.

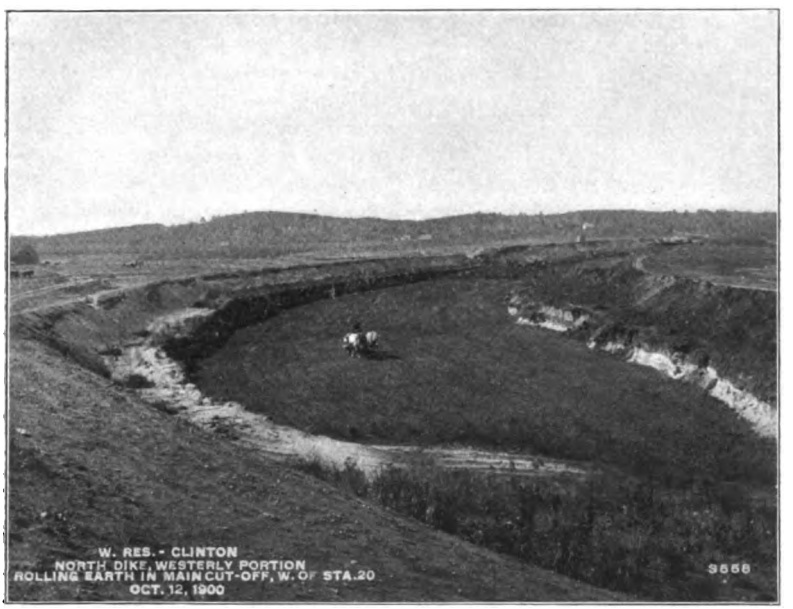

FIG. 2.—NORTH DIKE: FILLING CUT-OFF TRENCH WITH SOIL.

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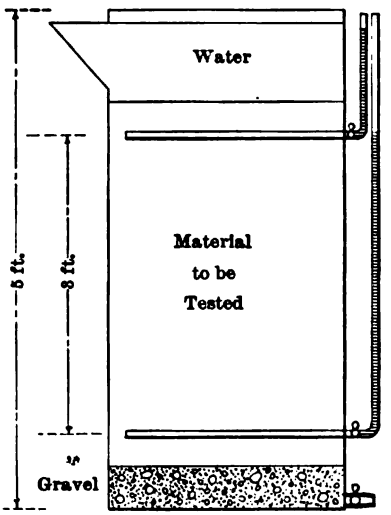
In addition to the filtration test, the experiments showed that a light section of dike, composed of soil not rammed, would withstand a head of 8 ft. of water without failure. Mr. Stearns

Filtration Experiments.—The experiments above cited show, incidentally, that extremely little water will filter through soil, even when it is not compacted, but a better test, particularly of the relative filtering capacity of soils and sands, can be obtained by filtering through cylinders of known area.



CAN FOR DETERMINING FRICTIONAL RESISTANCE.

FIG. 12.

For this purpose, five galvanized-iron cylinders, having tight bottoms, were provided (see Fig. 12), each 2 ft. 4½ ins. in diameter and 5 ft. high. The horizontal area, consequently, was 0.0001 acre. In the bottom of each cylinder there was a small wooden drain, in the form of a cross, leading to a faucet on one side of the cylinder. Over the drain were placed stones about 2 ins. in diameter, and on them smaller stones, diminishing in size, were added, until at the top of a 5-in. layer the material was either a fine gravel or coarse sand. Upon this layer, which would remove water freely, was placed the filtering material to be tested, to a depth of 3 ft. 8 ins. Within this filtering material, 4 ins. above its bottom and 4 ins. below its surface, were placed horizontal, perforated, brass pipes covered with wire gauze. and extending through the sides of the cylinder. These were connected with ½-in. vertical glass tubes. The water to be filtered was applied in a steady stream at the top of the filtering material, so that it stood about 10 ins. deep, and was maintained at a constant level by admitting more than would filter, allowing the surplus to pass over an overflow. When the experiment was in progress the water appeared in the glass tubes, and rose in each of them to a height due to the pressure of the water in the filtering material in immediate contact with the corresponding perforated brass pipe. The difference in the height of water in the glass tubes showed the amount of head lost by filtering the amount of water passing through the material. When it was desired to filter with a smaller loss of head than that due to a free outlet at the bottom of the tank, a stop-cock was partly closed, so, that the water in the glass tube connected with the lower brass pipe would rise.

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Mr. Stearns. Experiments were made with gravel, sand of all grades, and soil from various locations, such as upland and intervale.

The experiments confirmed those of Mr. Hazen, by showing that the loss of head was directly proportional to the quantity of water filtered, and substantially confirmed his conclusion that the quantity filtered will vary as the square of the diameter of the smaller grains of the filtering material. The smaller grains referred to are those which separate the finer 10% from the coarser 90%, and are the grains which, according to Mr. Hazen's experiments, determine the "effective size" of the material for filtration.

To give all the experiments in detail, with the results of the mechanical analyses of the materials used, would require more space than it seems desirable to use in this discussion. The experiments with coarse and fine gravel are, therefore, omitted, and those with sand are grouped together in four grades, in Table No. 6.

TABLE No. 6.—AMOUNTS OF FILTRATION, IN GALLONS PER DAY, THROUGH AN AREA OF 10 000 SQ. FT. OF DIFFERENT MATERIALS, WITH A LOSS OF HEAD OF 1 FT. IN 10.

Coarse sand, average of three experiments	2 200 000
Medium sand, average of six experiments	400 000
Fine sand, average of two experiments	90 000
Very fine sand, average of two experiments	7 200
Soil, average of ten experiments	510

In order to express the results with a convenient unit of area, 10 000 sq. ft. has been adopted for the filtering area, and a loss of head or slope of 1 ft. in 10.

It is of interest to note the very great difference between the amount of filtration through the coarse sand and the very fine sand, and also to note how much less water filters through the soil than through the finest sand. The ratio of filtration through the various materials, as compared with the filtration through soil, is more clearly shown if each of the figures is divided by 510, so that the amount of filtration through soil will be the unit, as follows:

Coarse sand	4 353
Medium sand	784
Fine sand	176
Very fine sand	14
Soil	1

There was some question, in view of the very small amount of water filtering through the soil, whether there might not have been some accumulation of air in its interstices, which diminished the amount of filtration, and it was therefore decided to experiment on

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the horizontal filtration of water through soil. For this purpose the large wooden tank used for the experimental dike, for a length of about 50 ft., was filled to the top with soil taken from an upland pasture. In order to retain the 50-ft. section of soil in place, and to allow the ingress and egress of the water without disturbing the soil, screens were placed about 55 ft. apart, and about 2½ ft. of porous sand and gravel were filled in between the screens and the ends of the section of soil. The soil was rammed in 3-in. layers, and special care was taken to prevent any water from following along the sides of the tank. At each end of the tank a space was left unfilled with earth, and one of these spaces was filled with water to a depth of 7.85 ft., while at the other end a free outlet was maintained. Mr. Stearns.

After the experiment had been continued for more than a month the amount of filtration, although somewhat variable, averaged about 1.7 galls. in 24 hours, indicating that the rate of filtration was as low as that obtained by the experiments in the galvanized-iron tanks. The tightness of this soil dam could be better appreciated by seeing it than by a statement of the figures. Although the section of dam was 6 ft. long and very nearly 8 ft. high, the leakage from the outlet amounted to only 32 drops in a minute.

To test the filtration through soil under greater pressure, a 10-in. pipe, to which a cover could be bolted, was used in the place of the galvanized-iron tanks, and the pressure upon soil 8 ft. in thickness was raised as high as 65 lbs. per square inch, equal to a head of 150 ft., and the results were not materially different from those deduced from the experiments in the galvanized-iron tanks.

As the soil to be used in the dike contained from 4 to 8% by weight of organic matter, the question arose as to whether, if this organic matter were to disappear, the soil would not become permeable. To determine this point comparative filtration experiments were made in the 10-in. pipe with soil as it came from the ground, and again after the organic matter had been burned out of it at the mining laboratory of the Massachusetts Institute of Technology.

Experiments were made for eight weeks with the natural soil and for eleven weeks with incinerated soil, with the result that about one-fifth more water filtered through the latter than through the former.

Stability of Soil.—The question having arisen as to whether soil when saturated would not be so mobile that it would not withstand the pressure of water, two experiments of the same kind were made; one with upland and the other with intervale soil. Each gave substantially the same results.

To make the experiments, the soil was filled into the large tank already described, and compacted so as to form an embankment 2 ft. wide on top, 6 ft. high and with slopes of 2 horizontal to 1 vertical on

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Mr. Stearns. both sides, as shown in Fig. 13. The embankment was then saturated by filling the tank on both sides of the embankment with water to a height of 5 ft. above the bottom. The top of the embankment was covered with 4-in. planks each 2 ft. long, and, running lengthwise of the embankment. On these planks was placed a 12 x 14-in. hard pine beam, 6 ft. long, extending across the tank. Two large jack-screws were placed between this beam and another securely fastened above it, and were operated so as to press the planks into the soil. The apparatus did not permit of an exact measurement of the pressure, but it was between 28 and 42 tons applied to 12 sq. ft., making the pressure probably about 3 tons per square foot. The 4-in. planks were pressed down into the soil a little more than 6 ins., but the embankment was so stable that there was only a very slight bulging at the sides a short distance below the water level, showing that there was enough friction between the particles of which the soil was composed to make it stable.

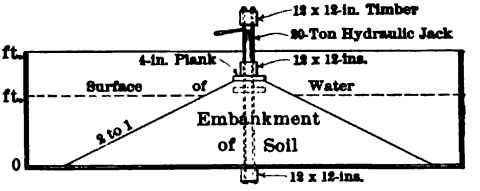

FIG. 13.

This experiment, in addition to showing the stability of the soil, was also interesting as showing the extent to which soil would compress under heavy pressure. Immediately below the planks, where the pressure per square foot was greatest, the soil became hard and compact.

The engineer in charge of the experiments states that his weight would easily push a sharp-pointed steel rod, $\frac{3}{4}$ in. in diameter, down to the bottom of the tank through the part of the embankment which had not been subjected to pressure, and that directly under the planks the soil was so compact that his weight would force it down only from 6 to 8 ins. This result is in line with other experiments subsequently made upon the compression of soil, and with practical experience where large banks of soil which have stood for a considerable time have been re-excavated. The compressed soil needs a pickaxe to loosen it, and is very different from loose soil such as one sees dug from the ground or dumped from a train.

Will Filtration Cause a Fine Material to Penetrate a Coarse One?—The question having been raised as to whether soil filled into a sand and gravel trench might, when the pressure was applied, penetrate the interstices of the sand, it was decided to make the following experiment, notwithstanding the fact that it is well known, from practical experience with water filtration, that fine particles are not carried to any considerable extent into the interstices of the sand. The 10-in.

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cylinder, already referred to, was filled to within 3 ins. of the top with medium sand and then 3 ins. of soil were put on the top; the end of the cylinder was closed, and water was applied, with a pressure of 65 lbs. per square inch. The water was allowed to filter through the soil and sand for a week, and upon examination at the end of this time there was no indication of any movement of the soil or that any of it had worked into the sand. Mr. Stearns.

Permanence of Soil.—An engineer naturally dislikes to use, in an important and permanent structure, a material which is not permanent, and the question, therefore, was raised as to whether soil which is known to contain a considerable amount of organic matter would be permanent. It was thought best to refer this matter to experts, and Mr. Allen Hazen and Professor W. O. Crosby were requested to report upon it.

They agreed in their separate reports that organic matter would disappear very slowly, indeed, except by oxidation with free oxygen present; that in the portion of the dike which was saturated there would be very little water filtering from the reservoir, and the free oxygen obtained from this source, therefore, would be very small; also, with regard to oxygen contained in the rainwater which fell upon the dike, that it would be removed from the water as it filtered through the upper layers of the soil. Their main conclusion, therefore, was that the process of oxidation below the line of saturation would be extremely slow, requiring many thousands of years for the complete removal of the organic matter, and that the tightness of the dike would not be materially affected by any changes which are likelyto occur. There will, no doubt, be some settling of the material of the dike in the course of years, mostly in the portions above the line of saturation, but, as due allowance for this settlement has been made in the height of the filling, it will do no serious harm. Below the line of saturation, which is the only place where changes in the condition of the soil can do any harm, there is a heavy pressure upon the soil, caused by the weight of the upper layers, and the material is sure to remain compact, even if some of the organic matter should be removed by chemical action. Mr. Hazen, in his report, suggested that this heavy load was likely to push the grains of mineral matter nearer together and to crowd the organic matter into the interstices, so that the disappearance of organic matter from the compressed soil would cause less settlement than from soil that was not compressed.

At the bottom of the deeper portions of the cut-off and of the maximum section of the dike there is a pressure of about 4 tons to the square foot upon the bottom soil, and of nearly a ton per square foot upon the soil at the limit of saturation. Attention has already been called to the experiment which showed that the soil with all organic matter removed was practically water-tight.

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Mr. Stearns. *Comparison of Dike with Existing Dams.*—Fig. 14 shows eight comparisons between the North Dike and existing earth dams. Where the existing dam had less height than the maximum section of the North Dike, the comparison is made with a section of the dike having the same height up to the water level; where the existing dam was higher than the North Dike, the comparison is made with the maximum section of the dike. The black, heavily-shaded portion is in every case the existing dam, while the light shading represents the North Dike. The black central walls shown in connection with several of the existing dams are clay walls, in every case excepting the Sudbury Dam, where the wall is of concrete, plastered on its up-stream face. Similar comparisons were made with many other dams, and they showed similar results, namely, that the dimensions of the North Dike are far in excess of those of any structure of this kind which has been built. The Jeypore Dam has no core-wall, and is composed largely of sand and other light earth, resting upon a foundation of the same general character.

Stability of Dams.—At this time, when not only the great dams of the Isthmian Canal are under consideration, but there is much discussion in the engineering papers with regard to the stability of earth dams, and many accounts of the failure of masonry dams, it seems opportune to discuss briefly the stability of dams in general.

In order that a dam may fail it is obviously necessary that the material of which it is composed must move, either as a whole or in part, under the pressure or the flow of water. If a dam has great bulk and weight and a reasonable degree of stability it cannot fail by sliding on its base or by overturning as a whole, but must fail by some separation of the parts.

In the case of earth dams, failures have generally occurred from the wearing away of the material by a current of water, either flowing over the top of the embankment or through some passageway formed in the dam, in connection with a pipe through it or by some burrowing animal.

Occasionally, an earth dam with steep slopes, or one with flatter slopes in which the material on the down-stream side retains the water, will fail through the sloughing of the down-stream slope, but the speaker does not know of any instance in which the earth confined in or under a dam has been moved by the filtration of water through it.

The tendency among many engineers at the present time seems to be to pay more attention to the materials and method of construction than to bulk. Many masonry dams have been built upon a somewhat theoretical cross-section, without taking into account the upward pressure of water on the base of the dam and the many uncertainties as to what will actually occur when the dam is put under great pressure; as a result, the failures of masonry dams over which large

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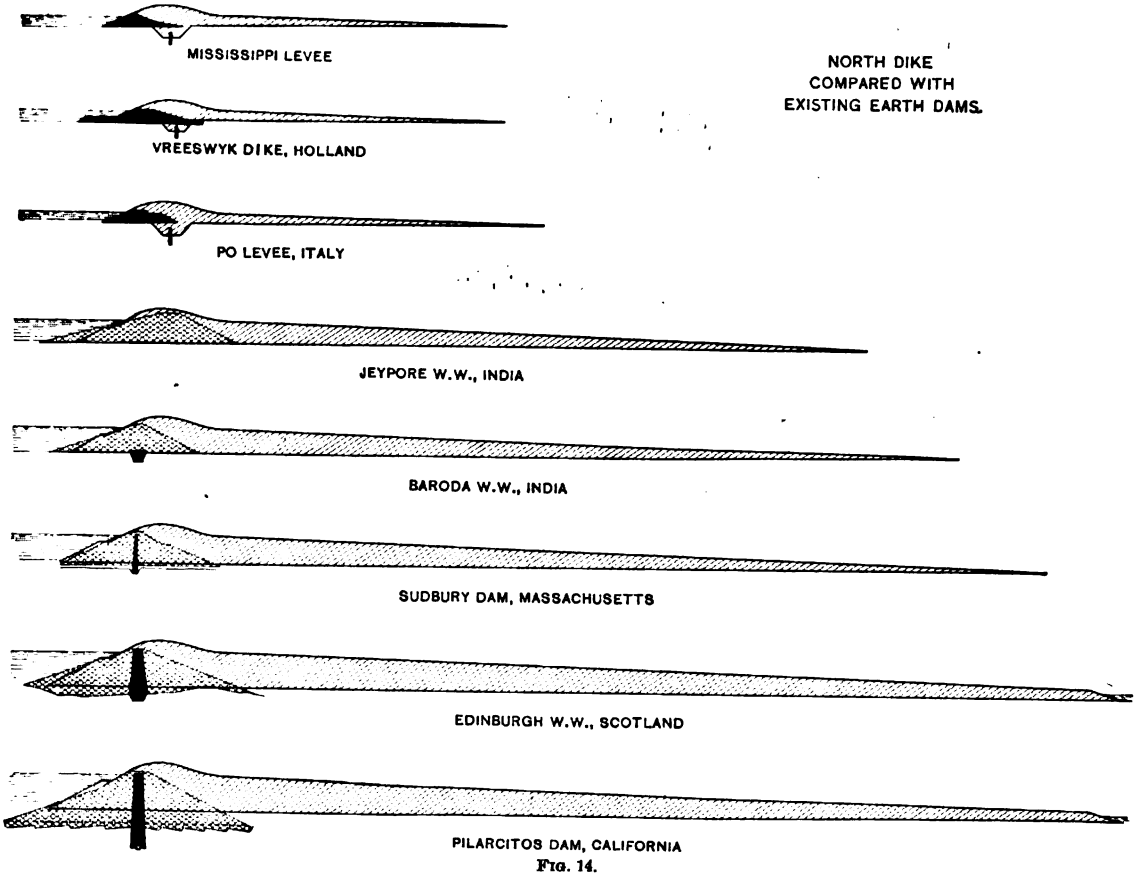

FIG. 14.

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Mr. Stearns. quantities of water flow have been numerous. A comparison of the sections of such structures with the sections of the upper part of masonry dams which rise to a considerable height above the water, like the New Croton Dam and the Wachusett Dam, shows the relatively small weight of these spillway sections, and no amount of good workmanship will compensate for the failure to provide sufficient weight.

The speaker has frequently been asked why he did not use Portland cement throughout the whole section of the Wachusett Dam, and his answer has been that, except in the parts of the dam where Portland cement has been used, natural cement will withstand all the pressures that can occur, and that with natural cement the same amount of money will build a dam of so much greater weight as to provide absolute security against sliding or overturning.

In the case of earth dams, the bulk is necessarily large, so that with well-constructed dams there are practically no cases of failure, except where the material has been washed away by a current of water.

In the case of the North Dike, the top being 15 ft. or more above the full-reservoir level and 10 or 12 ft. above any height which is likely to be reached during freshets, it is impossible that any water should ever run over the top. There are no pipes or structures through the dike to form channels through which any current of water could flow to remove the material. The dike is protected against burrowing animals by the paving and broken stone protection on the water face, extending 8 ft. above and 13 ft. below the reservoir level, but even without this protection an animal would have to burrow 289 ft. to reach a point on the down-stream side of the dike 3 ft. below the water level

There is another advantage in having a great mass of material above the water line of the reservoir, which is worthy of consideration, viz., that in the event of the brief control by an enemy or a mob it would be impossible to remove enough material to cause the failure of the dam.

The filtration through the dike will be so slow, and will take place where there is such a load upon the material, that there can be no possible movement of the particles on account of this filtration.

The precautions taken at the down-stream toe of the dike are such that the water cannot accumulate and cause it to slough, and even if any trouble of this kind should occur it would take place so far from the principal part of the dike that it could be remedied before any trouble could possibly occur.

In the case of the plan proposed by Mr. Morison for the Bohio Dam, the speaker believes that it cannot fail to be safe if the crest is

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built so high that floods or waves cannot possibly overtop it, and if the dam is properly built. Such a large mass of material, in a dam which is constructed so as to be less pervious near the reservoir than toward the down-stream toe, cannot be pushed or washed away. If a large amount of water should filter through the material under the dam and come out at the lower end, it may cause some movement of material in the vicinity of the toe, unless precautions are taken to prevent it. Few engineers who deal with filtration would wish to say that they could not design the down-stream portion of such a dam in a way to permit the water filtering through the dam to come to the surface without carrying earth with it. There are many large natural springs at the base of sand or gravel banks, and large quantities of water filtering into artificial excavations, which furnish examples of water coming to the surface without disturbing the material through which it comes. Mr. Stearns.

While referring to the percolation through such a dam, it may be well to call attention to the comparative sections of permeable material of the Bohio Dam and the North Dike, as illustrated by Mr. Morison in Fig. 8. The area of this material at the North Dike as shown by him is correct, and it is larger than the area given for the Bohio Dam. The degree of fineness of the material, however, is very different. He states that the effective diameters of certain samples from the Bohio Dam average by their squares 1.11 mm. Making allowance for these samples being coarser than the average sample of permeable material and for the fine material lost, he suggests that the average diameter of the material in place may be taken at 0.50 mm. The material classed as "permeable" at the North Dike has an effective diameter of about 0.20 mm., and, since the percolation varies as the square of the diameter, the percolation at the North Dike, with the same loss of head, would be less than one-sixth of the percolation per unit of area at the Bohio Dam.

The design of the North Dike was prepared after consultation with Joseph P. Davis and A. Fteley, Members, Am. Soc. C. E., and Mr. Hiram F. Mills, A. M., C. E., of Lowell, Mass.; and the final design received the approval of these engineers. Thomas F. Richardson, M. Am. Soc. C. E., Engineer of the Dam and Aqueduct Department of the Metropolitan Water-Works made most of the preliminary investigations, and Hiram A. Miller, M. Am. Soc. C. E., Engineer of the Reservoir Department, completed the investigations, worked out the details of the design, and has had charge of the execution of the work.

ALLEN HAZEN, M. Am. Soc. C. E.—The author has made use of a formula, for computing the flow of water in sands, which was originated at the Lawrence Experiment Station of the Massachusetts State Board of Health. This formula was established by certain experiments made Mr. Hazen.

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Mr. Hazen. to secure the necessary information for use in constructing filters, and particularly for use in constructing the Lawrence filter, which was just being undertaken.

The principal object of the Lawrence Experiment Station was to secure scientific information regarding the process of filtration. The applicability of this information depended upon the discovery of a sure method of comparison by means of which the materials used in the experimental filters could be compared with the materials which it was proposed to use in other filters, and which were so numerous that detailed investigations of them were impossible.

The formula as devised and quoted may be divided into several parts, some of which were new, and some of which had long been known. The discovery that the velocity of water in any given sand varies directly as the head, and inversely as the distance, is attributed to Darcy. It was carefully checked at Lawrence, and found to be correct. This knowledge was useful, but it was imperatively necessary, in addition, to know the coefficient for various materials. It was not enough to know that v would vary with $\frac{h}{l}$. It was necessary to know what v would be for any given material and slope.

The novelty of the Lawrence work in this direction consisted, first, in improvements in the methods of mechanical analysis of sands; second, in establishing a definition of the effective size of sand; and third, in finding the relation between this effective size and the velocity.

The problem was a somewhat complicated one. The ground was entirely new, and the information was required for immediate use. As the experiments were instituted and carried out, various forms of apparatus were used successively; but the final and most successful results were obtained from galvanized-iron cylinders containing the material under examination, supported by graded gravel layers at the bottom, through which a flow of water was maintained at accurately controlled and determined rates. The losses of head in determined distances, ranging from 12 to 48 ins., were determined by attaching glass tubes to pet-cocks in the sides, and observing the differences between the water levels in the various tubes. These observations were always confined to portions of the sand layer well below the surface, to avoid the additional head required to force the water into the sand at the surface, for this additional head was often greater than the friction in a considerable depth of sand. The appliances were such that the apparatus could usually be kept in continuous use for weeks or months; and the most reliable results, particularly for fine sands, were obtained after considerable lengths of time.

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Mr. Hazen.

The effective size, as defined at Lawrence, was that in which 10% of the material was finer and 90% of it coarser than itself, the size of each grain being taken as the diameter of a sphere of equal volume. Methods of analysis were developed to allow this determination to be made with considerable accuracy. The procedure of taking the effective size as stated was purely empirical. It was selected because it brought the materials used in the Lawrence experiments having different uniformity coefficients (that is to say, materials in which the amount of mixing of coarse and fine grains were very different) into their proper relations with each other.

In the nine years since this formula was originated, some additional data have been secured. Experiments by F. P. Stearns, M. Am. Soc. C. E., have furnished some of the best of these results. Altogether, these data have been less numerous than could have been desired, but they have been reasonably consistent, and have shown the correctness of the formula. Of course, the value of c varies somewhat with the conditions. On an average, it is somewhat less than was at first supposed.

The flow of water through sands has been investigated upon independent lines by Professor F. H. King,* of the University of Wisconsin, who has developed formulas which give, apparently, for a given sand, a somewhat lower discharge than is computed by the Lawrence method. Direct comparison, however, is difficult, because the effective size, as defined by Professor King, is an entirely different quantity from the effective size as defined in the Lawrence method.

Professor Philip Forchheimer, of Gratz, Austria, has made an interesting compilation of the experiments on the flow of water through sands,† beginning with Darcy's work in 1856, and coming down to the present time. He has computed and prepared a table of the values of c, in the Lawrence formula, as deduced from the experiments of four European observers, all ante-dating the Lawrence work. The values which he thus finds range from 310 to 1 110, averaging 442. This comparison is somewhat uncertain, because of differences in the methods of computing the grain size used in the various cases. Probably the effective sizes by the Lawrence method are always less than the grain sizes taken by the other investigators; and, if this is so, the factors found by Forchheimer from their results should be increased by the square of the ratio between the grain sizes as computed by Forchheimer and the effective sizes as defined by the Lawrence method. This might double the value of c, although Forchheimer writes that, in his judgment, the ratios of grain size to the effective sizes, on an average, would not be more than from 1.24 to 1.30, corresponding to an addition of from 54 to 69% to the values of c as computed by him. Taking this into account, the general agreement

* U. S. Geological Survey, 19th Annual Report, 1899.

† *Zeitschrift des Vereines Deutscher Ingenieure*, Vol. XXXXV, pp. 1736 and 1781.

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Mr. Hazen. of the experiments among themselves and with the Lawrence experiments is quite satisfactory; and, as Forchheimer states, the value of the comparison is greater because each of the observers was ignorant of the others, and none of these results was known to us at the time the Lawrence experiments were made.

As to the application of the formula to the computation of the probable flow of water through bodies of sand enormously greater than those used in the experiments, the speaker is unable to see any rational ground for objection to such use. In experimenting with pipes, the loss of head in 100 or 1 000 ft. of pipe, may be determined, and, assuming that the connections are made on the sides of the pipe sufficiently far away from the points of entrance and exit, no one would doubt that the rates thus found would be applicable to lengths of pipe indefinitely longer than those used in the experiment. The speaker believes that the same is true of sand, but with this difference, that the motion of water in sand is many times slower, the interstices are very much smaller, and all the actions are of such a nature as to establish themselves normally in a very short distance. The few feet of sand on which experiments have been made may therefore be quite as adequate, as far as length is concerned, as the lengths of pipe which have been used in most of the experiments to determine the flow of water in them.

The conditions of the sand in and under a large dam would probably differ somewhat from the conditions of the sand in the filtration experiments. The experiments made by Mr. Stearns, at Clinton, were more nearly comparable to the conditions in a dam, for the materials were usually placed in layers and rammed. Under these conditions, the values of c in the formula averaged about 600. Generally speaking, the sand in a filter is in the most favorable condition for the passage of water, and the value of c is at a maximum. The conditions in and under a dam, so far as they are different, all tend to lower the value of c, and, consequently, to reduce the amount of seepage.

Mr. North. EDWARD P. NORTH, M. Am. Soc. C. E.—It is, doubtless, a matter of pride to all that, after twenty years of varying plans proposed by the European engineers for the Panama Canal project, the last resource is in close conformity to the plan proposed before this Society in 1879 by Charles D. Ward, M. Am. Soc. C. E., to build a dam at Bohio and carry the summit level at approximately the same height as proposed by the Isthmian Canal Commission. But it is doubtful if Mr. Ward would propose a dam which necessitated work in caissons under an air pressure balancing a water column of more than 130 ft.

There seems to be an ambiguity as to the exact depth, as in one part of the Commission's report a minimum depth of 128 ft. below tide is mentioned, and another part of the same report reads that the cofferdam is to be 8 ft. above tide. As the dam site is some 13 miles inland,

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it is assumed, subject to correction, that the air pressure in the caissons will be that due to 136 ft. of water, or, virtually, 5 atmospheres. Mr. North.

It is assumed, and all will readily agree, that healthy men sustain small inconvenience when working 4-hour shifts under pressures of about 50 ft., but at 75 ft. deaths are to be expected, and at 100 to 110 ft. they occur with unpleasant frequency even among picked men; also, that in the Great Lakes 110 ft. is about the limit of divers' work. All these experiences have been had where the temperature of the water has been between 40 and 50° Fahr.; whereas, at the Bohio Dam site the temperature of the water is about 90°, as the mean annual temperature—which governs, approximately, the temperature of deep-seated springs—is about 90°, and further, this is the temperature assumed by Mr. Allen Hazen in his percolation formula for that locality. This high temperature in the caissons must add materially to the distress of those working in them. When, to these difficulties is added the fact that, as far back as the seventeenth century, maps have designated the Caribbean front of the Panama Route as "The Place of Bones," an evil reputation that has ever since been maintained, the necessity, as implied in this paper, for designing the least expensive safe structure adapted to the particular case in hand, is apparent. Unless the head of water in the Chagres River can be pumped down to reduce the pressure in the caissons so that life will not be in great jeopardy, a procedure that is probably practicable during low-water flow of the river, the saying of 40 years ago, "that each tie on the Panama Railroad cost a human life," will be repeated, as to the depths in feet to which the caissons are sunk.

Unfortunately for the advancement of engineering knowledge, a courteous regard for the expressed wishes of the author prevents any consideration here of the reasons for selecting a route requiring air-locked foundations more than 130 ft. deep, when about 80 ft. would suffice for a masonry dam at Conchuda; or why a rock and earth dam at Bohio should be more safe and available than at or below Ochoa and for the San Francisco basin; having in view the much larger flow of the San Juan, which reduces danger from seepage to a minimum.

As there seems to be no tenable doubt as to the stability of the proposed dam, there remains only for consideration the plan and its ability to retain sufficient water for navigation during the annual dry season, or through two or more recurrent dry years. On account of the limited water-shed, 875 sq. miles, difficulty may be apprehended in case there should be two successive dry years, even with the traffic estimated for, and if the canal is made large enough to conveniently pass the traffic between the east and west coasts of this country, which experience with the canals at the "Soo" shows may be expected, it is of vital importance that the seepage or any other waste should be reduced to the minimum.

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Mr. North. The speaker suggests that the dam, as shown in Fig. 6, if turned around, would more effectively retain water and would cost less.

The value of an enrockment at the base of an earth slope, as shown by Mr. Stearns, and as used in wet railroad cuts, lies in its preventing the degradation of the toe, which commences by small slides. Dams Nos. 3 and 4, Fig. 6, are requisite to save an indefinite extension of the earth slope, the great length of which is obviously not required for stability. Nor are Dams Nos. 1 and 2 of much economic use in preventing erosion by the feeble waves possible in Lake Bohio, if an embankment of like mass is to be used.

If Rock Dams Nos. 1 and 2 were retained at or near their present proposed locations and the 2 000 ft. of earth embankment were extended out into the lake it would effectually seal a large area on the bottom against the access of water to such portions of the pervious strata mentioned as do not underlie any extension of the heavy and tight blanket of finer alluvial deposit found at the dam site. Both the rock and earth portions of the dam would be preferably deposited in running water, obviating the cost of a temporary dam, ensuring the thorough filling of the rock dam and the puddling of the earth.

Any floods that might occur during the construction of the dam would save a great part of the excavation necessary for the proposed puddle dam. The depth to which the water passing over the dam would cut out the material below it, depends, of course, on the volume and velocity of the water striking the alluvial deposit, modified by the angle of its incidence. While there is small question that with a vertical fall the material would be cut out to the rock by an ordinary flood, the result would be different and less with the water running over the flat slope assumed by a loose-rock dam, particularly if material from the lock excavation was used instead of rock blasted for the purpose.

To whatever depth the cutting extended, the toe of the rock dam would be strengthened by its greater depth, and, on the completion of the dam, an easy and comparatively inexpensive problem in clam-shell dredging would be presented, by which an open ditch could be excavated to the rock and filled with clay to shut off all percolation after the stability of the slopes of the Cubebra Cut was established, and if it was ascertained that water percolated under the embankment.

Referring to Mr. Ford's suggested method of construction (page 307), a plate facing page 357 of the Report of the Chief of Engineers, U.S.A., for 1875, shows that his proposal is not entirely new, it being preceded by a plan devised by the late Col. Francis U. Farquhar, U. S. Engineer, M. Am. Soc. C. E., for preserving the Falls of St. Anthony by preventing the water of the Mississippi from cutting out the soft sand rock underlying the limestone cap at the crest of the falls. The completion of this work is mentioned on page 563 of the Report for 1877.

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Boyd Ehle, Assoc. M. Am. Soc. C. E.—Mr. Morison has covered the subject so thoroughly on its theoretical side that little can be said in addition. There is one point, not mentioned in the paper, which would be in favor of the construction proposed by the author at Bohio. It is probable that it would be safer in case of earthquake action than a masonry or a core-wall dam. This subject, however, received the careful consideration of the Isthmian Canal Commission, and was passed over as not being a serious danger. It may have been similarly dismissed in this paper. The recent earthquakes on the Isthmus, however, have cracked masonry and adobe walls, at times, quite seriously, and, in more severe shocks, might be a factor to be reckoned with in canal construction. Their action would have no more effect on the elevated plateau that Mr. Morison proposes than on any other part of the impounding reservoir. Mr. Ehle.

Mr. Stearns has shown an earthen dam, under a head of 70 ft., performing its duty efficiently in spite of the leakage. A similar case, on a somewhat smaller scale, occurred in the writer's early experience, in building an intake dam for a reservoir on a small stream. The channel was closed by a timber weir abutting on a sand and gravel point, in deference to the wishes of the Water Commissioners. Sheet-piling, driven lengthwise of the weir to a practically impermeable stratum, was quite effective, but there was considerable seepage around this and through the gravelly material, which was shown by large springs at the foot of the bank. This occurred under a head of about 12 ft. As there was plenty of water, and the bank held, anxiety was dulled, and the reservoir has served its purpose for a long time.

In the great irrigation reservoirs of India there are earth embankments, miles in extent and greater in height than Mr. Morison proposes. These were built by native carriers with baskets, and compacted by elephants, and they do their duty as surely as the fine masonry dams built by the English engineers. Given sufficient earth, there must be a limit to the size of a great mound which will impound water. This is the situation on the Panama Canal, where there is a quantity of material greatly in excess of that which Mr. Morison proposes to use, and which must be wasted. If it is once on the cars it can just as well be dumped in the Bohio dam, although there seems to be no doubt in the chain of reasoning that the dimensions proposed will be ample.

In regard to Mr. North's statement that the 90-ft. pressure on this bank of material might produce a movement that would endanger the dam, it would seem that this objection, if probable, would apply with much greater force to a light core-wall dam. Of course, by using pneumatic caissons, the dam would not be tight, either at the base or around the sealing caissons. It would seem to be impossible to sink them to the great depths proposed, on a straight line, as drawn on the plan; and if there was a movement, by earthquake or pressure, it

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Mr. Ehle. would be but an earthen dam, subject to seepage, but of very much less section than Mr. Morison's dam. It is not by any means granted that the core-wall will be troubled by pressure, and, without doubt, the dam, as planned, would be a safe and conservative design, but the earthen dam may perform similar duty and be more economical.

As Mr. Morison has located his dam far enough above the Commission's site, it would seem wise to build it, against all objections. It would be as cheap as any temporary dam, and it seems very probable that it would be as efficient. If any serious doubts as to safety arose, it would be of great service in building the Commission's dam.

A point, foreign to the paper, has been interjected into the discussion, in regard to the probable unhealthfulness in building the dam, on the basis of the statement, that "each tie of the Panama Railway has cost a man's life." To anyone acquainted with the conditions, this is a gross exaggeration. Citizens of the United States, long in the employ of the Panama Railway, seem to have no such knowledge of great mortality, and have no trouble with the climate. Whatever measure of truth there is in this story is due to the entire absence of any sanitary precautions or a proper water-supply system. The water carts in Panama often take their supply from wells that drain the yellow-fever burying grounds. Again, it must be recognized that the Panama Route, almost from the discovery of the Continent, has been a great highway, without a proper quarantine. It is very probable that, if the work is undertaken by the United States Government, the same strict regulations which were enforced at Havana, Santiago and elsewhere in Cuba would be equally efficient at Panama or Nicaragua.

Another point, also brought up, but foreign to the paper, is the question of relative economy of a dam at Bohio and at Conchuda, in Nicaragua. The site of the latter dam is very dissimilar to that at Bohio, being in a rocky gorge without any proper location for a separate spillway, such as the Gigante, so that the dam must act as a weir, and have controlling works on the crest for the regulation of the summit level. It would seem that the Commission's plan is the correct engineering solution. As has been pointed out, it is the duty of the engineer to adopt the most economical, effective, and safe tool for a certain purpose, not any one or an elaborate one.

It seems probable that the reason earthwork dams have been going out of use, as engineering has become more scientific, is on account of the lack of method in analysis. If this is so, the formulas given by Mr. Morison place this structure on the same plane as the retaining wall and others. Even in bridge and structural iron-work there is a considerable factor of ignorance.

Mr. Paschke. THEODORE PASCHKE, M. Am. Soc. C. E.—This paper is very interesting, and the plan proposed by the author for the construction of the Bohio Dam, while not entirely original with him, is in the line of

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the highest degree of rational engineering. The author intimates that the question of seepage may be the only vulnerable point in his plan, and invites discussion on this particular point. Mr. Paschke.

The Isthmian Commission's plan seems to indicate the same trend of thought, inasmuch as that plan involves carrying a core-wall to rock foundation at an unprecedented depth below water.

The speaker has had some experience in the tropics, especially in Central America, in the construction of water-works and river improvements, and, based upon this experience, he feels called upon to allay any apprehension or fear there might be on the question of seepage as regards this proposed construction of the Bohio Dam; and, however creditable the preceding discussions on seepage are to the gentlemen who so elaborately dwelt upon the subject, it strikes him that those learned discussions on seepage are really unnecessary, and not applicable to this dam at all, for the simple reason that, in the speaker's opinion, all that part of Bohio Lake in front of thedam will be silted up and made water-tight, for all practical purposes, in the course of a few seasons' work, by Nature herself.

The strata overlying the hills and mountains through Central America have as their base, almost exclusively, volcanic ashes of various compositions, according to the period in which they were emitted. Some have been hardened to a soft rock like sandstone, others have the appearance of fine loam, and others are not unlike burnt clay. All are easily eroded by water, and are carried down by every flood in all the mountain streams in large quantities. They are ground up and mixed thoroughly with vegetable matter, forming a black clay-like silt; a very good water-proofing indeed. The Gigante Spillway will divert the line of flow of the Chagres waters from the site of the dam, and will have the effect of forming a large eddy in the flow in front of the dam, which will hasten the process of silting up.

Indeed, the speaker is of the opinion that it is only a question of time when the whole artificial lake, which the proposed dam will form, will be filled with this silt, except the channel necessary for the Chagres waters to flow either to the spillway, during floods, or to the locks of the canal, and that, if no provision is made for such exigency, the ulterior object of creating this artificial lake will be defeated.

While the speaker most readily and heartily affirms all the advantages claimed by the author for his proposed plan of the Bohio Dam, he cannot help but deplore the fact that it is designed in the interest of only a lock canal across the Isthmus at that point.

Engineering ingenuity, of an order which can devise a plan to dam the Chagres at Bohio, at once so simple and grand, and so enduring, should find no difficulty in devising means to dam the same river a little higher up in its course and to completely control the flow of its waters at will, removing the only great obstacle in the way of a sea-level canal at this point.

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Mr. Burr. WILLIAM H. BURR, M. Am. Soc. C. E. (by letter).—Taking the North Dike of the Wachusett Reservoir of the Metropolitan Water Supply for the City of Boston as his pattern. in some of its principal features, Mr. Morison develops a rather striking plan, involving some highly interesting but novel and untried features, for the dam near Bohio on the Panama Canal Route. The physical features at the site of the proposed Bohio Dam are only partially set forth by the author, but it is essential, for the complete appreciation of the conditions under which this structure must be built, to exhibit all the results of the borings made by the Isthmian Canal Commission at the various tentative sites, of which there are six, Fig. 2 showing but five. The omission of Section *E*, a short distance up stream from *F*, is not of crucial importance, but that section is needed to complete the information obtained by the Commission, and aids one in understanding how greatly variable are the subsurface strata at and near the vicinity of the dam site.

In reality, that excessive and erratic variation in the strata of material disclosed by the borings is one of the most striking features of the whole situation. It exhibits clearly, among other things, the danger of drawing any conclusions as to the impermeability of the upper strata of material, which, as Mr. Morison indicates, are either clay, or sand, or a mixture of the two. Further, it is essential, in dealing with this general question, to realize fully that the order of the strata as found at one section may be radically different in one or more of the other sections; indeed, that is generally the case. Further than that, at the same elevation and in the same section, practically clear coarse sand may be disclosed by one boring and blue clay with little sand by another boring perhaps not more than 300 ft. away. Again, yellow clay and sand may be found at the surface of one of the same borings and at 128 ft. below the surface in the other. Indeed, the one persistent feature of the disclosures by the borings is the utter lack of uniformity of conditions on which any conclusions common to different sections can be based. The masses of material seem to be in the most irregular and irregularly limited strata or pockets. The sand is of all degrees of fineness or coarseness, from the finest to coarse gravel, and apparently from clear sand or gravel to all degrees of admixture of clay, both blue and yellow.

There are a few bore-holes in which the standing of the water appeared to indicate impermeable material. In fact, the pure clay and some admixtures of sand and clay may safely be considered impervious, but the limits of impermeable areas have not been defined, and cannot be predicted from the information gained by the Commission in its borings. The excessive variability of that feature is a part of the general variability to which reference has already been made as one of the prevailing physical features of the situation. The sections or tenta-

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tive sites along which borings were sunk by the Commission are lettered *A*, *B*, *C*, *D*, *E*, and *F*, *A* being the farthest up-stream section and *F* the farthest down-stream section. A stretch of about 1 300 ft. along the course of the river between Sections *D* and *E* contains but two or three bore-holes, and, consequently, the information as to it is meager, the reason for so few borings in that vicinity being the unpromising character of the locality for the purposes of the Commission. In Section *A*, the borings, about 350 ft. apart, are generally in clay, there being little clear sand disclosed. In that section, apparently, the river water is not in direct contact with the pervious substrata. The borings made at Section *B*, however, show the water of the river coming in contact with the heavy layers of sand disclosed by them, thus making a free flow from the river to the pervious substrata. In Section *C* the river bed is also of sand, and there is every reason to believe, from the results of the borings there, that the water may find its way through the mixture of clay and sand down to the heavy masses of clear sand reaching nearly down to the bottom of the geological valley. The same general observation may also reasonably be made in connection with Section *D*, the bed of the river being in a layer of clay and sand with clear sand underneath it. The borings made at both Sections *E* and *F* show that the river is in direct contact with the heavy mass of permeable sand extending practically to the lowest portion of the materials penetrated, or to rock. Mr. Burr.

In these observations, material which has been called sand is in some cases coarse enough to be classed as gravel. It will thus be seen that the total statement of the information actually disclosed by the borings at the various sites demonstrates conclusively a probability so strong as to amount to practical certainty that, not only is the water of the river now in free connection with the pervious substrata of sand and gravel, but that the water of the lake, with its 90 ft. additional head, would be in much freer connection.

Again, if it could be assumed that a blanket of impervious clay or mixture of clay and sand is uppermost over pervious material, it is in the highest degree probable that the weight of 90 ft. of water above it would in many places produce settlement cracks through which water would readily find its way to the pervious strata below. These physical features of the situation induced the Commission to believe that in the present state of information its estimates and plans should be based upon a structure which would close completely the geological valley, and thus shut off absolutely any possibility of dangerous or prejudicial seepage. In this connection it is to be remembered that it is not danger to the structure alone which is to be guarded against, but any degree of seepage which might prejudice the supply of water in the summit level of the canal formed by the lake.

The location at Section *F*, for the purposes of plans and estimates, was adopted by the Commission, as stated by Mr. Morison, for the

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Mr. Burr. reason that the maximum depth from sea level to rock was 128 ft., which is but 13 ft. more than the depth reached under one of the piers of the New East River Bridge. It is possible that the deepest caissons at Bohio might be carried to rock by the pneumatic process under the full head of 128 ft. This selection would perhaps appear more suitable if Mr. Morison had stated that the deepest rock at the site selected by the French engineers is 143 ft. below sea level. It is not impossible that that site might not eventually be adopted, but it was deemed best not to select a site for the purpose of plans and estimates against which the objection of depths far beyond those already reached under compressed air might be brought. Indeed, the writer is not sure that Section *B* might not prove to possess controlling advantages, as it is short, and the maximum depth of rock is less than 140 ft. It is possible, also, that the open-dredging method might be available for putting in place the deepest caissons, but it involves considerable uncertainty as to the closing of the lowest part of the geological valley, where the material might be most pervious, and, as already stated, the Commission desired to eliminate as far as possible all uncertainties, even at some additional cost.

Again, another material advantage offered by Section *F* arises from the fact that within the banks of the actual river the deepest rock is less than 100 ft. As shown by the elevation in Fig. 4, the remainder of the structure in the deepest part of the excavation could be built by sinking caissons on dry land, and mostly at a considerable distance from the water of the river. That portion of the dam, about 1 300 ft. in length, is situated along the high clay plateau on the left or westerly bank of the river. It is to be anticipated that water stands in the substrata at practically the elevation of the water in the river, or at least at sea level, but if pits are excavated in that plateau down into the pervious strata, ample experience shows that the water in them could readily be pumped down to a considerable amount, which the writer believes would prove to be not less than 30 or 40 ft., with reasonable pump capacity. Such depressions of subsurface water elevation, by the operation of pumping, have frequently been effected in engineering works; indeed, the ease by which this can be done has proved to be an objectionable feature of many water supplies drawn from subsurface water-bearing strata. It may confidently be anticipated, therefore, that by these simple and available means of pumping down the head of water around the caissons sunk throughout the deepest portion of Section *F*, the head of water producing pressure in the air-chambers of the pneumatic caissons may be reduced below 100 ft.; indeed, the writer believes it would prove to be materially less than that.

It is to be observed that the head inducing the flow into the pits subjected to pumping would vary only from zero to the greatest de-

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pression of the water surface, and that the area of percolation would be small for any one pit. The seepage or percolation through the great area of section of pervious substrata would always take place under the high net head of 82 to 90 or 92 ft. Mr. Burr.

It is thus seen that the construction of the dam on the site adopted by the Commission, for the purposes of its plans and estimates, involves no new or untried processes or features more novel than those due to naturally varying local conditions. In other words, this construction is quite within the limits of engineering work already accomplished, so that the estimates of the Commission for this dam cannot be attacked successfully. It is not certain that it would be necessary to carry pneumatic caissons the entire distance to bed-rock in this location. If a depth were reached within a short distance, perhaps 10 or 20 ft., of that rock, or if small, deep places should be encountered, there are various methods, including that of sheeting or other devices, by which the seepage of water could be positively and completely shut off below the cutting edge.

On page 258 of the Commission's Report occurs the following portion of a paragraph:

"A less expensive dam at Bohio has been proposed, but through a portion of its length it would be underlaid by a deposit of sand and gravel pervious to water. The seepage might not prove dangerous, but the security of the canal is directly dependent upon this dam, and the policy of the Commission has been to select the more perfect structure, even at a somewhat greater cost."

It is barely possible that a more detailed examination of the vicinity of the dam would demonstrate the feasibility of the type set forth by Mr. Morison, although the writer doubts it, but if a structure of that character had been adopted by the Commission its plans would have been open to serious criticism as involving elements of uncertainty to a degree that cannot be measured or predicted.

The writer has not been able to confirm the low estimate which Mr. Morison places upon his proposed earth dam, and is inclined to think its cost would be nearer $4 000 000 than $2 500 000. Whether or not that be so, however, the additional estimated cost involved in the Commission's plans is abundantly justified by the absolute certainty in the character of the results obtained by it in connection with a feature positively essential not only for the safety of operation of the canal but for its actual existence.

The computations of seepage made by Mr. Morison can scarcely be relied upon as even approximate, for the reason that the data on which the formula used are based were derived from experiments on masses of sand almost infinitesimal compared with those under consideration, and also upon conditions which it is practically certain are radically different from those existing at Bohio. That formula applies only to

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Mr. Burr. sands having effective diameters ranging from 0.1 to 3 mm., and with a uniformity coefficient lower than 5. It also presupposes that the texture, so to speak, of the sand is uniform. It is not at all unlikely, in such material as that which exists at Bohio, that there may be places so free and open as practically to form small channels. In fact, there is no basis for any prediction as to the existence of conditions which would make it even approximately accurate in this connection to use a formula derived from experiments in small filters, even though that formula be admirably adapted to all other filter conditions.

The type of dam which Mr. Morison has developed and set forth certainly is most interesting, but, under conditions existing at Bohio, it involves novel and untried features, the efficacy of which can be established only by further engineering experiences, as well as assumptions which yet remain to be justified.

Mr. Menocal. A. G. MENOCAL, M. Am. Soc. C. E. (by letter).—Mr. Morison deserves the thanks of the engineering profession, and also of all those interested in reaching a satisfactory solution of the Isthmian Canal question, for his timely and valuable paper on the proposed dam at Bohio, on the route of the Panama Canal. The dam, together with the lake created by it, and the Alhajuela Reservoir, as sources of water supply, are of vital importance to the success of the plans recommended by the Isthmian Canal Commission, and the three are so intimately connected with each other that the failure of either of them to accomplish the essential requirements would throw serious doubts into the practicability of the project. They should, therefore, receive the most attentive consideration, and all doubtful elements should be removed before a route for building the canal is finally selected.

The difficulties involved in the execution of the Commission's plans, as shown by physical conditions developed by a partial examination of the site, are pointed out clearly in the paper; but those difficulties are likely to be enhanced greatly by a more detailed survey, or during the process of construction. Holes deeper than 128 ft. below sea level may be found, or faults or joints discovered in the rock, where the caissons may have to be carried down to greater depth than 136 ft. below low water, before a tight joint can be secured. The writer is not aware that the pneumatic process has ever been carried to such a depth, and, considering the attending labor and climatic conditions, is of the opinion that it cannot be done successfully at Panama. The scheme seems to be in the nature of a hazardous experiment, containing in itself several elements of failure.

The alternate plan, combining rock-fill dams and earth filling, proposed by Mr. Morison, has the advantage of simplicity in the design and economy of time and money in execution, but, in the opinion of the writer, would not meet the requirements of the problem. Such a dam may safely stand the normal pressure of 90 ft., and, with a per-

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manent superabundant supply of water, would probably maintain the proposed level in the lake, but it would not prevent a much greater leakage than can be spared from the water supply available at Panama, even when this supply is enlarged by the proposed reservoir in the upper valley of the Chagres River. The borings made in the vicinity of the dam site, according to the Commission's report, show: Mr. Menocal.

"A variety of material; hard clay, soft clay, sand, and some mixtures of sand, clay and gravel in varying proportions. These materials are found in beds of varying shapes and thickness, not distributed with uniformity and not arranged according to any general law from which can be deduced the characters of the soil at points other than those actually examined. In every section constructed from the borings, strata of greater or less dimensions are found, which are permeable by water."

It is quite evident that ground composed of material as above described cannot be regarded as impervious under high pressure, and the difficulties of driving through it two long rows of sheet-piles to the depth of 60 ft. so as to secure uniformly tight joints, must be admitted. Many of the piles are likely to break or split, while others will be deflected laterally, making large, open joints underground, without marked indications at the tops. Considerable leakage must be expected, therefore, not only through the pervious material represented in the sections accompanying the paper, but also through the whole submerged area, through the joints, and underneath the sheet-piling. Much of this leakage will come to the original surface and flow freely along the imperfect joints made by the ground and the material loosely deposited between the rock-fill dams.

There seem to be no formulas by which this leakage can be computed. That deduced by experiments by Mr. Allen Hazen, on filter beds with small heads and selected material, is not applicable to the conditions of this problem, as a small change in temperature or in the diameter of the sand grains would give widely divergent results, tending to check faith in the conclusions.

The amount of leakage permissible through the dam is directly proportional to the available water supply, and it seems proper, in this connection, to look into the capacity and permanency of the storage reservoir provided in the plan for operating the canal during the dry season. The Chagres River is the only source of supply, and Lake Bohio, with an area of 38 sq. miles, the main storage reservoir. Another reservoir is proposed to be created in the upper valley of the river by building a dam at Alhajuela, about 10 miles above Obispo.

The Chagres has a well-established reputation as a violent torrential stream. Its flow has been known to vary from a minimum of 350 to a maximum of 136 000 cu. ft. per second. It has been known to rise 35 ft. at Obispo, where the canal falls into the bed of the river, and to rise 39.3 ft. at Bohio. The excessive rainfall and the precipitous slopes of the water-shed, rising in places to thousands of feet,

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Mr. Menocal. give to the river its torrential character. The records of the last twenty years show that these extreme high floods take place every two or three years, but similar floods of somewhat less violence are not uncommon, and are likely to occur several times during the same rainy season. It is stated in the Commission's report that, above Obispo the Chagres "is in general a clear stream flowing over a bed of gravel; but sand, clay and silt in moderate quantities appear in the lower portion of its course." This statement must have reference to conditions during the dry season, as it is above Obispo where the mountainous water-shed and an annual rainfall of about 130 ins. give the river its torrential features, and where the largest erosion takes place, as shown by the gravel beds referred to, and cliffs from 50 to 200 ft. high on either side of the river, alternately as the current has washed first one and then the other in its tortuous course. Anyone who has seen the river in floods knows that its waters are surcharged with sediment. Lieut. J. T. Sullivan, U. S. Navy, in his report to the Navy Department, in 1883, says:

"The flood waters of the Chagres are not only heavily charged with detritus, but with an immense debris as well. Houses, trees, masses of vegetation, rocks and stones are borne in its floods, and in a freshet which occurred in the winter of 1879–80, an iron tank 17 ft. above the level of the railroad track was carried away."

Referring to the rapid silting up of the sea-level section of the canal excavated by the French Company, Alfred Noble, M. Am. Soc. C. E., member of the Isthmian Canal Commission, stated before the Committee on Interoceanic Canal of the Senate that "the only practical solution would be to keep the river waters out (of the canal). Dredging would certainly be too expensive."

In the presence of these well-known conditions, it seems that the necessary result of receiving the river in the proposed Bohio Lake will be the rapid silting up of the lake and the consequent decrease of the water supply; that an attempt to create a reservoir in the upper valley of the river will, also, for the same reasons, fail to accomplish its purpose, and that the water supply cannot be relied upon as sufficient for the purposes intended, or permanent.

Mr. Pharr. H. N. PHARR, Assoc. M. Am. Soc. C. E. (by letter).—"The third plan," proposed by the author, has been of special interest to the writer. In the construction and maintenance of levees on the Mississippi River, along the front of the Lower St. Francis Levee District, in Arkansas and Missouri, about 200 miles in length, embankments solely of earth have been built to withstand greatly varying heads of water, under various and particular conditions and considerations, which have necessitated different heights, dimensions and methods of construction.

The writer's observation and study of the action of earth dams under various conditions and circumstances and of greatly differing

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character have extended through many years; and the character of the work at Bohio, and the conditions which will govern the building of an earth dam of such dimensions as described by the author are thought to be appreciated. Mr. Pharr.

Although nothing within the writer's work or observation is reasonably comparable with the work and conditions described in the paper, there are some features in particular cases which may be considered.

The levees as generally constructed have an 8 to 10-ft. crown, with slopes of 1 on 3, to 1 on 4, sometimes with broken slopes, or with banquettes of varying height and dimensions. A levee of 20 to 25 ft. in height, to withstand a flood or water height of 2 ft. below the crown, is very usual. However, a particular location or levee (Chute of 38) is presented in Fig. 15, which may not be considered usual, though practically the same conditions and features have been met with several times along the levee line. A general map of the Mississippi River in the vicinity of the Chute of Island Number 38 is shown in Fig. 15. The main or navigable channel of the river has in the not very distant past occupied several beds in this vicinity, as shown by the islands and "old rivers" now deserted by the river and becoming silted up and grown up in cottonwood, willows, etc. The bend west of Centennial Island, known as Devil's Elbow, was cut off and abandoned in May, 1876.

The map does not show the exact shore and channel lines as at present existing, owing to caving which has recently taken place along the east side of Centennial Island and the west side of Fogleman Chute. The caving at the latter is one of the considerations which determined the location of the levee across Poker Point and back from the river bank as shown. The levee above and around Old River was located safely close to the bank which was sloping and free from caving at that point, the old river being now in process of filling up.

Island Number 38 is shown on the map as now a part of the mainland, though long ago the main channel of the Mississippi was on the west side of it. The Chute thus abandoned on the west side of Island Number 38 is known as the Chute of Island Number 38, and, locally, as Little Old River, but it has been silted or filled up practically by the sediment and overflow of the river. It is scarcely observable at its upper end. The lower end has been very greatly contracted, but was kept open to a certain extent by Frenchman's Bayou, which affords the only natural drainage for a considerable territory. It was the leveeing of this chute, to prevent the ingress of the floods of the Mississippi River and the diversion of the drainage formerly afforded by Frenchman's Bayou, which had to be secured.

The character of the material, of course, is all permeable, being alluvial deposit, a sandy loam; and the foundation or base of the levee

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Mr. Pharr. immediately across the chute was of a soft muck. An iron rod or pole could be pushed down, merely by hand, into the bottom or sides of the chute for the length of the rod. Water, to the depth of a few feet, was always running through the chute into the river when low or during dry seasons, but when the river was rising the current ran back through the chute, to again reverse and flow into the river as it fell. Low water in the chute was about 8 or 10 ft. above low water in the Mississippi River and in Old River.

On Fig. 15 there is also a profile, on the center line of the levee across the chute, with an exaggerated vertical scale, showing also the grade line, which was raised 2 ft. across the chute to allow for any additional settling which might occur, after the levee was once brought up to grade, beyond that taking place during construction and beyond the usual allowance of 10% for shrinkage, and also to prevent overtopping at this immediate point. Overtopping was likely to occur to the general levee in this vicinity, as the available funds for construction limited the height which could be adopted for the stretch of levee, some 17 miles in length, built at that time.

A cross-section and a plan of the levee, as constructed across the chute, the latter being drawn to the same scale as the profile, are shown in Fig. 15.

On this figure there is also a cross-section of the levee, drawn to true scale, with details of height, distances, slopes, etc. The net height is 35 ft.; crown, 8 ft.; side slopes, 1 on 4, on both river and land side; with an off-set or banquette on the land side, 20 ft. in width and with a slope of 1 on 20. The elevation of the banquette is about the same as the bank on each side of the chute, or the natural surface of the adjacent ground, which is practically a level plain. No core or puddle-wall or sheet-piling was used or constructed, the principal object being to obtain an embankment of as great homogeneity as possible, because the foundation and ground were permeable. Three excavations or muck ditches, however, were dug across the chute, one just on the river side of the center line of the levee and one each near the middle of the front and rear slopes, as far as the material excavated would allow. These ditches were merely for purposes of exploration. The whole ground having been built up by the sediment and sand of the river, it was to be expected that many logs and other perishable matter would be found throughout, but only such as could be discovered near the surface by such means was undertaken to be removed. The ditches were about 8 ft. wide and 6 to 8 ft. deep where the material would allow. Other excavations were made in the base in removing perishable material.

The levee was constructed according to the general levee specifications, as far as applicable. The method of carrying up the embankment continously in layers of a certain thickness and well traveled was

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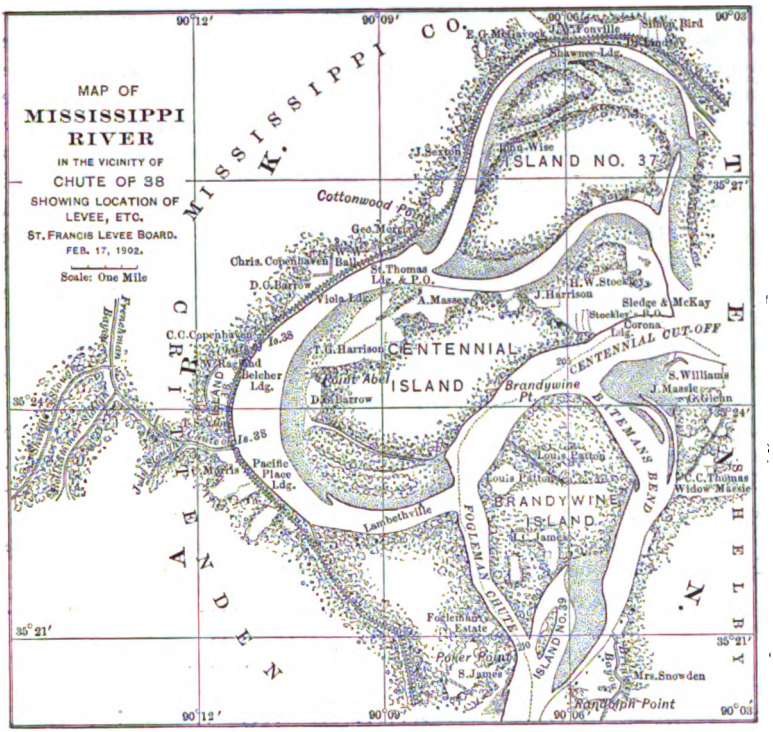
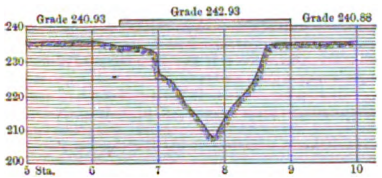
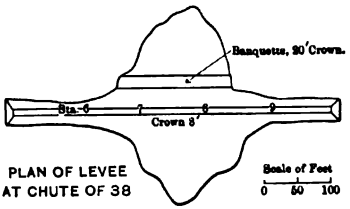
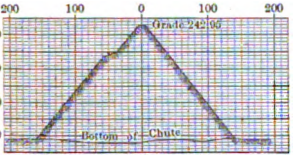




PROFILE OF CHUTE OF 38

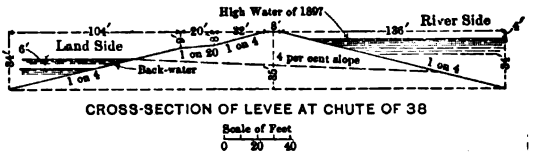


PLAN OF LEVEE AT CHUTE OF 38



CROSS-SECTION OF LEVEE AT CHUTE OF 38

Scale: Vertical, 1 = 5 Horizontal.



CROSS-SECTION OF LEVEE AT CHUTE OF 38

FIG. 15.

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Mr. Pharr. complied with, to a great extent. Wheel scrapers with about 35 teams were used. At the time, the chute was discharging a stream about 4 ft. in depth into the river with considerable velocity. No effort was made to dam the stream in any way above the levee site, as such would have required about as much time and expense as the proposed levee. The work was begun by depositing earth as far down the bank and as close to the stream along each side as the natural slope of the loose and new earth would permit without being carried away by the current. This was continued until a considerable quantity of earth was deposited toward the toe of the land slope of the levee, the surface or slope of the layers of new earth declining toward the river side. An almost imperceptible sliding or slipping of the whole mass and of the material of the foundation toward the center from each side was seen to be going on. This gradually narrowed the width of the stream, raised the surface of the back-water and increased the velocity of the flow until the new earth began to undermine and the sliding toward the center increased. Finally, the sides came together and effectually dammed the stream, which soon rose to a depth of 10 or 15 ft., but the embankment was easily kept above it as the teams could then cross. The embankment was completed in December, 1898, without interference, according to the dimensions shown by the cross-section in Fig. 15.

The following shows the total number of cubic yards placed in the levee across the chute proper:

Embankment and excavation	21 786.2	cu. yds.
Additional excavation	1 600.0	"
Settling of foundation	3 500.5	"
Total	26 886.7	cu. yds.

This, under a contract for the construction of 17 miles of levee, cost 12.45 cents per cubic yard, or $3 347.39.

The specifications allowed a free haul of 300 ft., but, as borrow pits were obtained upon the four sides of the levee and chute, no overhaul was necessary. Probably a fair estimate of the cost of the levee across the chute only, under separate contract, would be 30 cents per cubic yard, or $8 066.

The surface of the water in the bayou or chute on the land side of the levee continued to rise until, in June and July, 1899, which was a very rainy season, a considerable territory was submerged by the back-water, which caused damage and inconvenience. The first natural relief was drainage through several bayous running in a southerly direction from Frenchman's Bayou and discharging into the Wampanocca Lake, which drains naturally toward the rear, finally reaching the St. Francis River. However, there was no relief in this direction until the water surface in the chute rose to Elevation 227, when escape began through Bellhammer Slough, located as shown in Fig. 15. The

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elevation of the base of the levee being 208, there was then a depth of more than 19 ft. continually standing against this levee. The maximum width of the base of the levee was 308 ft., and the thickness of the embankment at Elevation 227, the surface of the back-water, was about 158 ft. Mr. Pharr.

Under these conditions the front or river slope of the levee was dry, and practically no seepage occurred.

Bellhammer Slough was canalized and deepened, somewhat more than a year after the construction of the levee, and the elevation of the surface of the back-water standing against the levee was lowered about 6 ft., or to Elevation 221, where it now stands with scarcely any variation. Practically no seepage takes place, and the chute between the levee and Old River is being rapidly filled up by the sediment and deposit of the river.

As formerly stated, the levee in this vicinity is not thought to be of sufficient height to restrain the maximum flood of the Mississippi River, and during the present year it is proposed to raise the grade 3 ft. and to enlarge and strengthen the levee proportionally.

So far as the writer's experience and study extend, there appears to be no reason why dams constructed of earth should be limited to any particular height, provided the slope and dimensions are sufficient. In the plan proposed by the author, the slope, base and dimensions certainly seem to be sufficient to withstand safely the head of 90 ft., the dam finishing 10 ft. above the level of the water impounded in the lake. The 4% slope, for material of that character, is no doubt much flatter than the slope of saturation. Probably the material in the foundation and for the dam at Bohio is more permeable than that generally obtained for the construction of levees in the Mississippi Valley, and the constant head in the one case might not be reasonably compared with the duration of the floods of the Mississippi River, though they stand sometimes as long as from 40 to 60 days within a few inches of the top of the levee; however, the incomparable dimensions proposed it seems would more than overcome the difference in conditions. It may be expected that such a dam and foundation will become less permeable as both the embankment and foundation settle and become more compact, and that seepage will decrease.

It appears to the writer that it might be advisable to flatten the slope on the down-stream side of the high temporary earth dam, proposed in the third plan, from 1 on 2 to at least 1 on 3. This, however, would increase its cost considerably, which as proposed is nearly as much as the estimated cost for the permanent dam. The suggestion to undertake the construction of the permanent dam without the construction of the temporary dam is worthy of very serious consideration.

From a comparison of the Commission's plan with the third plan,

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Mr. Pharr. if it may be considered that the slight seepage in the latter will be of no practical disadvantage, the very great difference in the cost, say $6 755 095 for the Commission's, and $2 025 675 for the third plan, should be a very strong inducement and justification for the adoption and construction of the latter. There is, of course, no material, out of which a dam can be constructed, of as great a permanence as earth.

Mr. Duryea. EDWIN DUBYEA, Jr., M. Am. Soc. C. E. (by letter).—Mr. Morison states that the criticism made on his plan is that "it permits seepage through the permeable sand and gravel in the lower part of the geological valley."

His plan makes no attempt to prevent or diminish this seepage, and he contents himself with showing that the amount of water thus lost is relatively unimportant. It is just possible, however, that the actual amount of seepage might prove to be much larger than he believes can be the case, and it is this possibility which seems to be the objectionable feature of the plan. This objection will be removed if any practicable plan can be found which will prevent or diminish the amount of seepage. Such a plan will be most reassuring and most effective if it is of such a nature that it can be applied either before or after the construction of the dam, so as to be used only in case the seepage proves to be greater than is thought advisable, and if the plan is so flexible that it can be applied to as small or as great a degree as seems necessary, or can be resorted to at any subsequent period.

It is believed that a plan combining all these favorable features is that of forcing cement grout into the layer of permeable sand and gravel by means of pipes driven through the overlying materials and through the dam itself. The ordinary method of making wash-borings makes it practicable to reach the permeable layer quickly and cheaply, and the fact that the layer is permeable and allows water to rise in the pipes makes it certain that grout under pressure can be forced into it for some distance away from the pipe. The top and bottom of the layer of sand and gravel should be first located by the boring; then, beginning with the foot of pipe at the bottom of the layer, the grout should be pumped under pressure while the pipe is being raised by small increments to the top of the permeable layer. The pipe could then probably be withdrawn at once. Both the success attending the operation and the proper distance apart at which to place the holes could be ascertained either before or after the construction of the dam by observing the action of the grout in open pipes driven at various distances from the pipe being grouted. The efficiency of the plan in preventing seepage could be tested by completing a line of grouted holes across the valley and then testing the difference in height to which water would rise in pipes driven on the up-stream and on the down-stream sides of the line, or by completing two lines

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of grouted holes some distance apart up and down stream and then testing the height to which water would rise in pipes driven between the lines. Mr. Duryea.

Any number of holes could be driven, but it seems that a double row arranged so that they form the apexes of equilateral triangles of 8-ft. sides should be quite effective. This would make a hole for each 4 ft. in width of the permeable layer. The width of this layer, on the sections shown, is nowhere as great as 300 ft., so that an allowance of 100 holes should be ample. There should be no difficulty in driving these holes and doing the grouting effectively for the sum of $500 per hole. This would make the cost of the 100 holes $50 000, or—allowing, say, 200 holes for safety—a cost of $100 000 for a plan which would remove the chief objection to Mr. Morison's proposed design. A means to positively control the seepage would be cheap even at twice this cost.

Mr. Morison's plan is very original in that it removes by the most radical means all danger from saturation. Past experience, however, has not shown this danger to be a very real one in ordinary earth dams of good design and construction. There seems to be no reason why his dam should cover such a large area, except that by this means the volume of rock in the four rock-fill dams is perhaps kept down to a smaller amount than otherwise. His design is very economical, and, if more complete examinations show no worse conditions at the site than are now believed to exist, should probably be adopted in its original, or in some modified, form.

The object in view by the Isthmian Canal Commission, however, was probably to present a plan which would meet the worst conditions that might be developed by more thorough examinations, and at the same time to assume as a preliminary estimate a cost so high that it would cover any reasonable plan which might be finally adopted. While all efforts should be made to use a more economical plan, such as that of Mr. Morison, it was certainly only prudent and conservative for the Commission to adopt for their purpose the much higher cost of their proposed design.

C. A. Sundstrom, M. Am. Soc. C. E. (by letter).—This paper is valuable and instructive, and if the United States Government decides to complete the Panama Canal, Mr. Morison's plan should be adopted in preference to any other, on account of increased utility, minimum cost, least amount of skilled labor required and completion in the shortest time. His plan offers another great advantage, in that it can be executed without the use of pneumatic caissons. Pneumatic caissons are very objectionable when they must be sunk to an unprecedented depth, in a tropical country, where the temperature is seldom, if ever, below 90°, and when, after the completion of the work, no better result has been obtained than if another, and less drastic, method had been adopted. Mr. Sundstrom.

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Mr. Sundstrom. The object of the Isthmian Canal Commission's plan, so far as the writer can see, is to reduce the seepage as much as possible, by constructing an earth dam, enclosing a core-wall, which is to be carried down to bed-rock by means of pneumatic caissons. Admitting the possibility of sinking caissons to the required depth, it must necessarily be a most difficult problem to fill the space between them with impervious material. It can be assumed, with reasonable certainty, that this can be done only within certain limits; that is to say, a depth will be reached, below which the spaces between the caissons must be left open and cause a seepage fully as great as that which may occur through the dam proposed by Mr. Morison.

The nature of the permeable strata, which cover the bed of the valley to a considerable depth, is the same at the site of the Isthmian Canal Commission's dam and the one proposed by Mr. Morison. Consequently, the same factors for calculating the velocity of the percolating water can be used in both cases, the length of the percolation being the only variable quantity. Mr. Morison's calculations show a velocity of 0.002 ft. per second under his dam, and a seepage of 40 cu. ft. per second. Assuming the length of the strata through which percolation will take place at the Isthmian Canal Commission's dam to be 500 ft., the velocity of the percolating water will be

$$V = \frac{1000 \times 90\ (90 + 10)}{500 \times 60}\, d^2$$

$$V = 300\, d^2 \text{ m. per day, or}$$

$V = 0.0117$ ft. per second, when d is assumed to be 1 mm.

There are thirteen openings, each 4 ft. wide, giving a length of 52 ft. along the dam, through which seepage can pass without interruption. In order to produce a seepage of 40 cu. ft. per second, the depth, d, of the percolating strata will be found to be

$$52\, d \times 0.0117 = 40,$$

$$\text{giving } d = 65.7 \text{ ft.}$$

The elevation of the bottom of the core-wall is — 128. Consequently, the depth to which the percolation must be interrupted will be $-128 + 65.7 = -62.3$.

This shows, beyond a doubt, that in order to obtain the same result as Mr. Morison, the Isthmian Canal Commission, after the caissons have been put in place and the core-wall has been constructed, have a Herculean task ahead of them. If they succeed in solving the problem that far, they will have a structure which will cost $4 700 000 more than Mr. Morison's plan, will require a great amount of skilled labor, and will consume considerably more time in its construction.

Mr. Wegmann. EDWARD WEGMANN, M. Am. Soc. C. E.—(by letter).—The plan proposed by Mr. Morison for the Bohio Dam is so much more economical than either the plan of the French engineers, or that recommended by the Isthmian Canal Commission, that it certainly deserves very careful consideration.

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Mr. Wegmann.

A dam built in this manner would certainly be water-tight itself, and the only leakage that might occur would be due to percolation through the permeable material below the proposed sheet-piling. Such leakage, if it did not exceed the amount estimated by Mr. Morison, would be of no consequence.

In constructing dams for impounding water for a domestic supply, it is generally necessary to make them so tight that practically no water is lost, except by evaporation. In storing water for irrigation or for supplying a canal, such water-tightness in a dam may not be necessary.

If Mr. Morison's estimate is even approximately correct, the loss of water by percolation under the dam would be relatively unimportant. We may doubt whether Mr. Hazen's formula for the flow of water through a uniform layer of sand is applicable to a mixed layer of sand, gravel, etc., such as probably exists under a part of the proposed Bohio Dam, but, on the other hand, there is no question that there would be a silting up which would steadily diminish the loss by percolation.

Should the loss of water from this cause prove to be much greater than that estimated by Mr. Morison, so as to impair seriously the usefulness of the reservoir, this loss could be diminished very materially by sinking wells, by dredging, or, if necessary, by pneumatic caissons on the down-stream side of the proposed dam, or in the earthen part of the dam, a little up stream from the down-stream rock-fill. If the latter plan should be adopted, the earth filling at the place where the wells or caissons would have to be sunk might be omitted until the remainder of the dam was completed, when experience would show whether the percolation had to be checked. The expensive work of sinking wells or caissons could thus be postponed until it was found to be absolutely necessary.

The water percolating beneath the proposed sheet-piling, which is to extend 50 ft. below tide, would move with a very slow velocity, have little head, and could certainly be checked at the down-stream side of the dam.

Wells sunk by dredging, as suggested by Mr. Morison, placed as closely together as possible and filled with concrete would probably suffice for this purpose. Grout might be forced between the wells, as low down as a diver could work, to check still more the percolation. If it were deemed necessary, pneumatic caissons could be sunk on the down-stream side of the dam, but even in this case, Mr. Morison's plan would probably be cheaper than that proposed by the Isthmian Canal Commission, as he has adopted a shorter location, needs no concrete core-wall in the dam, and proposes to construct the dam mainly of waste material from excavations made for the canal.

Of course, future borings may show reasons for modifying the opinion expressed herein, but the additional facts that may be ascertained are just as likely to be for as against the plan proposed by Mr. Morison.

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Mr. Forchheimer.

PHILIPP FORCHHEIMER* (by letter).—The formula used by the author for the calculation of the seepage is backed not only by Mr. Hazen's experiments but also by those made by Seelheim, Hagen and Kröber. For a temperature of 10° Cent., Mr. Hazen puts $V = 450$ to $1\,200\, d^2 \frac{h}{l}$, d being the effective diameter of the sand. The proportion of the mean diameter of grain, d_m, to the effective diameter, d, varies very much. For $d_m = 1.73$, d_m or $d_m^2 = 3d^2$. Mr. Hazen's formula would show $V\frac{l}{h}$ to lie between 112 and 400 d^2_m, or the relation, $V\frac{l}{h} : d^2_m$, to be between 112 and 400. According to the other experimentists, the corresponding figures are as shown in Table No. 7.

TABLE No. 7.

	Seelheim.		Hagen.	Seelheim.	Kröber.	Seelheim.	Kröber.			
d_m = ...	0.16	0.23	0.28	0.48	0.54	0.68	0.7	0.9	1.35	2.1
$V\frac{l}{h} : d_m$ = ...	323	319	310	328	367	324	358	425	314	355

If the sand is but slightly soiled by other substances the velocity diminishes very much. Professor Masoni, for instance, used different sands taken from the seaside near Naples, and from Mount Vesuvius. He did not wash his sand as thoroughly as Mr. Hazen, and Table No. 8 shows that most of his velocities were much smaller in comparison.

TABLE No. 8.

Sand.	I.	II.	IIa.	III.	IV.	V.	VI.
Effective diameters, in millimeters.....	0.55	0.45	0.45	0.46	0.40	0.38	0.37
Masoni's speed for head, h, equal to distance, l, in meters per day.....	231	9.2	22	47	40	15	11
Speed, in meters per day, according to Hazen's formula, $V = 1000\, d^2 \frac{h}{l}$, h being $= h$.....	302	202	202	212	160	144	137

If sandy soil contains clay to such an extent that the clay fills up the interstices between the grains of sand entirely, the percolation is so insignificant as to render the compound practically impervious.

* Professor of Hydraulics at the A. D. K. K. Technischen Hochschule, Grats, Austria.

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Seelheim made some accurate experiments on the percolation through sandy clay consisting of pure clay, powdered chalk, quartz sand and water (Al_2O_3, $SiO_2 + 2.3\ H_2O$). According to his formula, as rectified by the writer, the rate of flow, at a temperature of 12° Cent., in meters per day, is only Mr. Forchheimer.

$$V = \frac{1}{1\,000} \times \frac{0.0011\ \text{clay} + 0.0023\ \text{chalk}}{\text{water} + \text{sand} + \text{clay} + \text{chalk}} \times \left(\frac{\text{water}}{\text{clay} + \text{chalk}}\right)^2$$

the words clay, chalk, etc., meaning the volumes of the ingredients mixed.

In Nature, clean sands or gravels are rare, and the permeability of alluvial deposits is therefore mostly a slight one. Public water supplies are generally taken from the most permeable strata that can be got at. The investigations made for water-works are therefore of special interest. Table No. 9 gives the velocities (taken for the entire cross-section of soil) which ground-water of about 10° Cent., or 50° Fahr., acquires at a gradient of $h : l = 90 : 2\,500 = 0.036$, in some of the underground drifts studied for public supply :

TABLE No. 9.

Town.	Lyons.	Strassburg.	B. Gladbach.	Augsburg.	Vienna.	Bukarest.
Speed, in millimeters per second.....	0.068	0.179	0.099	0.148	0.084	0.051
Speed, in feet per second.....	0.000224	0.000588	0.000292	0.000485	0.000112	0.000166

At 90° Fahr., speeds are two-thirds higher than they are at 50°; but, even at that high temperature, the speed in none of the quoted grounds would come near to the 0.002 ft. per second taken into account by the author. Besides, it must be noted that some of these grounds contain gravel and pebbles some inches in thickness. It is thus apparent that to use Mr. Hazen's formula, which refers to well-washed sand, means to be on the safe side, as indeed the author wishes to be.

But there is one more danger to be feared, and that is the formation of springs. This depends entirely on the uniformity of the Bohio subsoil. Natural passages, in more or less impervious soil, filled with permeable material, are dangerous, if they are much narrower at the outlet than in the interior and near the water-basin. As the same volume of water flows through the whole length of such a drift, its velocity will be inversely proportional to its cross-section. The whole head of water will always be used for the flow, and but little of it will be lost where the cross-section is considerable and the movement slow. Thus the main part of the head of water will be left for the

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Mr. Forchheimer.

outlet, causing the water to gush out of it and wash away the sand. It is difficult to say beforehand whether or not the washing out will proceed far enough into the interior to constitute a real danger for the dam. It may be well to add that furrows cut into impervious earth and filled with pervious sand are to be dreaded too, if the water flow enters with a big cross-section and issues with a very reduced one, because the speed increases with the reduction of the cross-section. All these dangers are diminished exceedingly by the great breadth of the proposed embankment. Besides this, there is another well-known fact that gives security, and which has been pointed out by the author: It is, that turbid waters clog their beds with their deposits.

As the writer had to occupy himself with a question of seepage of some importance two years ago, he thinks that some figures about it will be of interest. They refer to the seepage from the river Lech into the discharging canal of the electric works of Gersthofen, in Bavaria. The soil consists of a bed of tertiary *flinz* covered by gravel. The former varies in its character between loam and loamy sand, and, although some water-bearing veins occur, it may be considered as practically water-tight. The gravel is pervious, and the writer—using Fossa-Mancini's trough—found the frictional resistance to be

$$\frac{h}{l} = \frac{1.77}{10^3} V + \frac{3.18}{10^4} V^2$$

V being the velocity, in meters per day. The surface of the *flinz* was slightly inclined, and showed some undulations or furrows. When a great part of the excavation was finished, the writer tested his calculations. The strip of land between the river and the excavation was 250 ft. broad. He calculated a seepage of 91.4 cu. ft. per second for a length of 13 500 ft. of the canal, whereas he only measured 46.8 cu. ft. per second. Part of the difference between the results of calculation and measurement was due to the fact that the gravel had been less closely packed in the trough than it was in its natural state; but the principal reason was probably the clogging of the river-bed by sediment.

According to the foregoing formula, used for the seepage at Gersthofen, the friction in gravel increases at a higher rate than the speed. This fact agrees with the experiments or calculations of other authors, as Smreker, von Welitschkowsky, Masoni and Hazen. The writer tested the adaptability of formulas having the shape

$$\frac{h}{l} = a V + b V^2,$$

as well as formulas having the shape

$$\frac{h}{l} = m V^n;$$

a, b, m and n being constants. He found the first form to correspond more accurately to the facts in Nature than the second one. The

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difference between the two formulas, however, proved to be only a slight one as regarded the Gersthofen material, and the writer's researches about it resulted in the figures shown in Table No. 10. Mr. Forchheimer.

TABLE No. 10.

V, in meters per day	10.7	27.5	52.6	64.7	67.3	90.2	100.3
$\frac{h}{l} = \frac{8.2}{10^4} V + \frac{1.07}{10^5} V^2$	0.010	0.031	0.073	0.107	0.098	0.161	0.190
$\frac{h}{l} = \frac{4.422}{10^4} V^{1.316}$	0.010	0.035	0.081	0.107	0.112	0.167	0.190
$\frac{h}{l}$, measured	0.010	0.030	0.070	0.107	0.110	0.170	0.190

The writer is not aware as to whether any of the subsoil of the Bohio dam contains material of such coarse structure as to be calculated by a formula of the shape

$$\frac{h}{l} = a\,V + b\,V^2,$$

but he would like to call attention to this possibility.

J. L. Campbell, M. Am. Soc. C. E. (by letter).—In 1896 the writer had occasion to carry an irrigation canal across the Rio Grande, in southern New Mexico. This canal is 34 ft. wide on the bottom, and is designed to carry 5 ft. of water. The river at the point of crossing, in the apex of a sharp bend, is 300 ft. wide and was carrying about 500 ft. per second during the low-water season, when this work was done. The entire Rio Grande Valley has a fertile alluvial soil, from 2 to 6 ft. thick, underlaid throughout by a deep bed of fine sand, thoroughly saturated to within about 10 ft. of the surface. The river channel is composed of fine sand and quicksand, with some fine gravel, all taking a slope of 4 or 5 horizontal to 1 vertical when saturated. Mr. Campbell.

The elevation of the grade line did not admit of an overhead crossing, and the canal was taken under the river-bed in four wooden pipe lines, each 50 ins. inside diameter. The tops of the pipes were placed from 3 to 7 ft. below the bottom of the channel, between five parallel rows of piling, cut off and capped over the pipes below the bed of the river. All this necessitated an open excavation across the river, 400 ft. long., 25 ft. wide on the bottom, and having an average depth of 10 ft. To do this, a temporary channel for the river was first cut across the neck of land formed back of the crossing by the bend in the river, after which a dam of sand bags was thrown across the channel parallel to and immediately above the site for the crossing, and made tight by dumping sand on the upper side. This forced the water through the temporary channel, and the excavation for the pipe lines began. This was accomplished as an open excavation by the use of "slip" scrapers, the sand being deposited against the dam

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Mr. Campbell. above and also formed into a dam across the channel below to prevent overflow from back-water, as the river rose.

When the excavation was completed, the side slopes had flattened of their own accord to 1 on 4, due to the saturation of the fine round sand. The water in the river stood against the upper dam 15 ft. above the bottom of the excavation, and 10 ft. above against the dam on the lower side.

The water was kept out of this excavation, which had a filtering area of approximately 40 000 sq. ft., under an average head of 12.5 ft., by two vertical centrifugal pumps, one a No. 8, and the other a No. 6. The No. 8 pump could easily keep the pit clear of water when running at its normal capacity of 400 cu. ft. per minute. Taking the filtering area of the pit at 40 000 sq. ft., and the mean effective head of water at 6.25 ft., the velocity of inflow, in feet per second, for the 90-ft. head on the Bohio Dam is as follows:

$$\frac{400 \times 90}{40\,000 \times 6.25 \times 60} = 0.0024 \text{ ft.}$$

As evidence that the maximum capacity of the No. 8 pump was greater than the inflow, there was an interruption of the pumping for a day, after which the pump succeeded in emptying the pit and keeping it empty, the No. 6 pump being out of service.

In making calculations for a large storage reservoir in the bed and valley of the Rio Grande, the writer has assumed that any loss of water due to filtering into the bed and sides of the reservoir would steadily decrease, due to the filling of the voids in any sand-bed by the fine silt carried by the river settling and being forced into the sand by the heavy pressure.

New irrigating ditches in the sandy soil of this valley lose much water when first put into service, but soon become comparatively water-tight by a coating of silt deposited by the water. At the end of an irrigating season, a ditch with a light grade will have 3 or 4 ins. of pure sediment covering its perimeter. The writer has observed that it requires several hours for water to soak through 6 or 8 ins. of this fine clay silt after it has been thoroughly dried, and that percolation through wet strata of it in the river channel is inappreciable.

If the Chagres carries as much silt as the Rio Grande (from descriptions of the former the writer concludes that in this respect both rivers are quite similar), the silting of the reservoir behind the Bohio Dam would certainly tend to a decrease of the percolation of water through all reasonably fine sand and gravel beds.

A tight row of sheet-piling, 40 or 50 ft. deep, under the dam, would make the filtering slow and difficult. The conditions of a filter-bed specially prepared to pass water at given rates are more favorable, and will give a higher rate, than the indiscriminate mixtures forming the structure of alluvial valleys. Proof of this is found in the fact that even filter-beds rapidly become clogged, and require constant cleaning.

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J. T. FORD, M. Inst. C. E. (by letter).—It is obvious that the dam proposed by Mr. Morison is, *per se*, superior in stability to the dam proposed by the Commission, not only because of its greater mass, shorter distance across the valley between abutments, and form of construction, which assures homogeneity throughout and extraordinary cheapness, but also because the masonry core-wall, especially that part of it which comes above the present natural ground line, in the Commission's plan, divides the whole structure into two disconnected wedges of earth, subject to possible different degrees of subsidence on opposite sides—the upper part of the core-wall may possibly crack under unequal pressures—and, if reliance is placed mainly on that core-wall for tightness, the results may possibly be disappointing, from a variety of causes, and, under extreme conditions, the danger point may even be reached long before such conditions could affect or even threaten the safety of Mr. Morison's dam. Mr. Ford.

The writer proposes a masonry apron-wall of peculiar construction, absolutely sealing the geological valley at about the site of Section *C*, *i. e.*, the French dam site, the narrowest point of the valley, and extending only up to the natural ground level, not into the structure of the dam at all, but simply as an additional or complementary feature of the

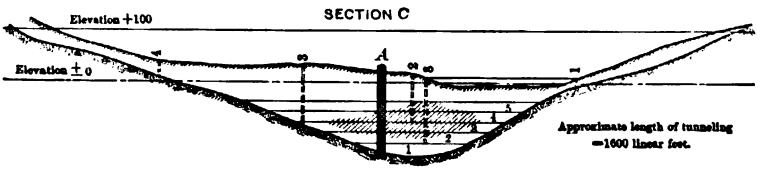

FIG. 16.

first-class structure proposed by Mr. Morison. See Fig. 16, the cross-section of the valley at *C*. The method here proposed for constructing this apron-wall is entirely novel, as far as the writer knows.* A good working shaft is sunk at *A*, Fig. 16, at any desirable distance up the bank and away from the edge of the water at normal river level; this shaft should go down to bed-rock, and its upper end, above ground, should be suitably protected, if necessary, from extraordinary flood levels. A tunnel (No. 1) is then started at the bottom of the shaft in both directions in the alignment of the proposed dam, of suitable dimensions, and timbered in the ordinary way, the bed-rock being followed and effectually sealed as the basis for the first tunnel floor. If the seepage is great the excavation can be filled as soon as timbered in comparatively short sections, or, if too much seepage exists for pumping, it is possible to use the freezing process for driving the tunnel, or even the extreme necessity of an air-lock carried forward at the end of the tun-

* See last paragraph of discussion by Mr. North, p. 282.

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Mr. Ford. nel would in such a position still give advantages over the present pneumatic methods, for cheap handling of material to be removed, boulders, etc., and, most important of all, an absolutely perfect sealing of the bed-rock. A gallery along the center of the tunnel would be left with such dimensions as would permit of effective handling of the material in and out to the shaft, pipes for the pumping and ventilation, etc., and, when completed, the galleries, if desired, could be filled with solid concrete, but it would seem to the writer a very great advantage indeed to leave the galleries open for permanent inspection of the work at all times. The jointing of the successive sections of such a tunnel would not be either smooth or perpendicular, but preferably be left with their natural slopes, after complete ramming, to receive the filling for the next section, which would thus give a continuous and satisfactory line of hollow wall, perfectly tight, both in its bed joints and section joints; it would also be desirable to leave sufficient space, below the roof timbering of the tunnel and above the concrete, for the proper ramming and depositing of the material.

Tunnel No. 2 would then be started from Shaft *A*, the roof timbering of Tunnel No. 1 being removed and the top of Tunnel No. 1 replacing the bed-rock as the floor for Tunnel No. 2, and so on, in succession, until such height is reached, or such conditions found as to seepage, etc., when it might be more convenient, or cheaper, to break ground at the top, with caisson and ordinary pneumatic work, and sheet-piling in the ordinary way, to carry the wall up to the original ground surface. Thus the most expensive and difficult part of the core-wall would be built under water without disturbing the overlying strata or letting in the water at all, and without assuming any risks of flood or other damage during the protracted periods of early construction. The valley would be sealed with a perfect and continuous wall, instead of a series of semi-detached masonry columns; or, to express it in other words, a wall with horizontal bed-joints would be built, instead of the so-far unprecedented vertical bed-joints proposed in the Commission's plan. The result would be a wall capable of inspection at any and all times during and after construction.

Few words need now be said as to the cost. The problem is simply the construction of 1 600 linear feet of tunnel, only differing from any ordinary railway tunnel in that the hole through the middle is smaller and that successive sections of the tunnel are on top of each other instead of being continuous from the end. What, then, will be the cost of 1 600 ft. of such tunnel, under the conditions described as obtaining at Bohio? The writer ventures to assert that he would make money on the contract at an average cost of $250 per linear foot. Thus the cost of 1 600 ft. would be about $400 000. Assuming that price to be doubled, and made $500 per linear foot, the question is simply this: Is it worth the sum of $800 000, in addition to the

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$2 430 810 total for Mr. Morison's excellent dam, to place that structure beyond cavil or risk of any kind, and thus keep the total cost of that great structure at a figure but little more than one-third of the total cost of the less desirable type of dam proposed by the Commission? Mr. Ford.

GEORGE S. MORISON, Past-President, Am. Soc. C. E. (by letter).— In closing the discussion the writer feels that it is only right to express his gratification at the manner in which this paper has been received and the general tenor of the remarks which it has brought out. Mr. Morison.

The *third plan* was purposely put forward in the boldest form, with no provision to limit seepage, as it was the wish of the writer to have the dam considered on the assumption that no efforts were made to shut off the seepage of water in the permeable earth below. It was not so much his purpose to show what might be done as to raise the question of the measure of danger which would exist if nothing were done. While it is not good engineering to spend for any purpose a sum incommensurate with the results achieved, neither is it good engineering to neglect inexpensive methods which may be expected to reduce loss. In actual construction, the writer would expect to make use of simple methods which would be likely to reduce the flow of water through the permeable material in the bottom of the valley. Various plans suggest themselves. One would be to drive a diaphragm of iron or steel sheet-piling, which, with the aid of a water jet, could be driven to a very great depth, possibly to the underlying rock. The method proposed by Mr. Ford is original, and would prove effective when done, although the writer believes it would be necessary to resort to the use of compressed air, and, perhaps, even to the freezing process, and that its cost would be much greater than Mr. Ford has estimated. The method proposed by Mr. Duryea, of grouting the sand with a cement paste forced down through pipes under pressure, was considered by the writer, and he believes that it would prove both economical and effective; his present feeling is that he should use this device if he actually had to construct the dam. It was the aim of this paper, however, to show that, however desirable such additional protection might be, it was not really necessary, and that a dam could be built which would be absolutely safe and economically water-tight, without the use of any unusual methods, and which would require a minimum of skilled labor.

The particular form of dam adopted by Mr. Stearns for the North Dike, with the high rounded crest, which, while costing but little, gives additional security against waves breaking over the dam, is probably an improvement on the section adopted by the writer, but it would be much less important at Bohio than at Wachusett, simply because the location of the Bohio Dam at the end of a narrow waterway at some distance from the broad lake is one in which the waves are less likely

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Mr. Morison. to be violent. Mr. Stearns' studies and work in this connection are of a character which the engineering profession cannot appreciate too highly.

Mr. Campbell's calculation, giving a speed for the flow of water through the gravel and sand practically identical with that calculated by the writer, is, perhaps, a coincidence rather than more. In this calculation no account is taken of the difference in the coarseness of the materials, or of the distance through which the water must travel. In all probability the sand in his work was finer than the sand which the writer has estimated on, and the length through which the water must pass was probably considerably less, the two balancing each other.

The writer takes issue with the statements made by Mr. Burr, that "there is no basis for any prediction as to the existence of conditions which would make it even approximately accurate in this connection to use a formula derived from experiments in small filters, even though that formula be admirably adapted to all other filter conditions," and by Mr. Menocal that the formula "deduced by experiments by Mr. Allen Hazen, on filter beds with small heads and selected material, is not applicable to the conditions of this problem, as a small change in temperature or in the diameter of the sand grains would give widely divergent results, tending to check faith in the conclusions." It is perfectly true that Mr. Hazen's formula, like all other hydraulic formulas, is derived from experiments on a comparatively small scale. On the other hand, it is no less true that formulas which cannot be used to determine actual results can be safely used to determine maximum results. The writer has never contended that his calculations made with this formula were close indications of what the actual seepage would be. He does insist, however, that they show a maximum seepage, which maximum he believes to be many times what the actual seepage would prove. In this connection, the words of Professor Forchheimer are exactly to the point. His Table No. 8 shows the rapid reduction in the passage of water through unwashed sand as compared with clean sand of the same effective diameter, while his Table No. 9 shows velocities which are but a small fraction of that which the writer has estimated upon. The sand under the Chagres River is unwashed and undoubtedly mixed with much foreign matter; the sands analyzed by Mr. Hazen had all been washed by the water in which they were brought from the bore holes.

Fig. 17 shows the results of the borings made on or near the proposed dam site. That these borings show some irregularities is undoubtedly true, but they indicate that there is generally permeable material in the lowest part of the valley. The conditions under which a given cross-section would pass a maximum amount of water would occur when the position of that permeable material was perfectly reg-

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PANAMA CANAL.

BOHIO DAM.

DIAGRAM OF BORINGS.

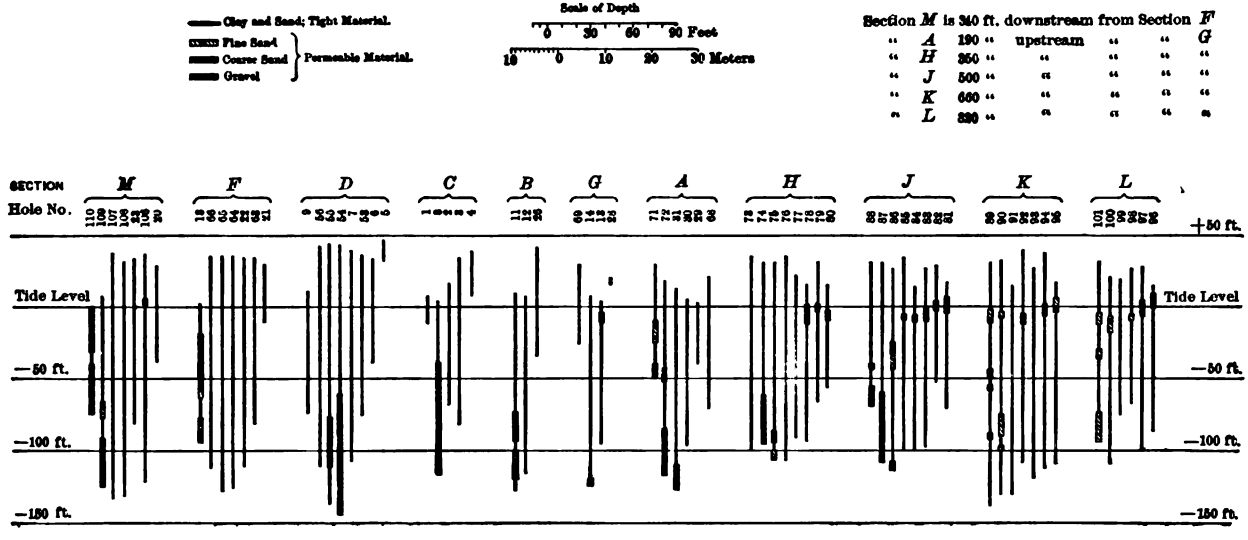

FIG. 17.

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Mr. Morison. ular so that it formed a straight and direct body of sand; the writer has assumed this material to be in such position. If, however, it is as Mr. Burr states, distributed "in the most irregular and irregularly limited strata or pockets," the difficulty which the water would meet in passing through would be aggravated by such irregularities, and the seepage would be reduced. The total length of borings shown in Fig. 17 below tide water is 6 075 ft., of which 816 ft. indicated permeable material. The total cross-section of the valley at Section *C* below tide water and above rock is 59 400 ft. If the permeable portion of this section bears the same relation to the whole that the permeable length of borings bears to the whole length of borings, this permeable section would be 7 960 sq. ft., or 40% of the 20 000 used in the calculations.

It must be remembered that under a given head the same amount of water will pass through 10 sq. ft. of sand, 5 ft. thick, as will pass through 100 sq. ft., 50 ft. thick. A thin diaphragm with a moderate number of holes placed in a large bed of sand would have comparatively little effect in reducing the flow through that sand; a thick diaphragm with the same area of holes would have a much greater effect. All the water which it is estimated would pass through this bed of sand, of 20 000 sq. ft. cross-section, 2 500 ft. long, would flow through a single orifice of less than 0.6 sq. ft. section in a thin plate. The actual thickness of the core-wall proposed for the Commission's dam, measured from out to out of sealing of cylinders, does not exceed 20 ft.; openings aggregating 160 sq. ft. of section, and filled with the same permeable material that is found under the river, would pass as much water as the whole 20 000 sq. ft., 2 500 ft. long. The total length of joints between caissons, below tide water, is 1 430 ft., and, as the cylinders have to be jointed to both caissons, the total length of joint is 2 860 ft.

Mr. Burr proposes to reduce his head for the pneumatic work by lowering the surface of the water from 30 to 40 ft. around the caissons, which he believes could be done with reasonable pump capacity. If this can be done, it is the best possible evidence that the seepage will not be serious. To reduce the water pressure, it is not only necessary to lower the water immediately around the caisson, but to take out enough water to reduce the head of saturation by 30 or 40 ft. Unless this is done, although the surface of the water may be lowered, the head of water against which atmospheric pressure must be provided will not be reduced. While the area of percolation immediately surrounding the pit would be small, it would increase rapidly in all directions, so that at a distance of 100 ft. from the pier on either side it would probably equal the entire cross-section of the valley, and the amount of water which would percolate through the short length of limited section near the pier would be as great as would percolate through the whole permeable portion of the valley

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beyond. Furthermore, as the level of saturation must be carried down below tide-water, seepage will occur from both directions, which means that the amount of pumping will be doubled. The writer has estimated on a possible maximum seepage of 40 cu. ft. per second in one direction, under a head of 90 ft. This would correspond to a seepage of 13.33 cu. ft. under a head of 30 ft. This last figure is equivalent to 800 cu. ft., or about 6 000 galls. per minute, or 8 500 000 galls. per day, and to 17 000 000 galls. daily if allowance is made for seepage from both sides of the dam. In view of the fact that while sinking the caissons there will be no dam in existence, and no broad base to compel the water to pass for a long distance through the bed of sand, it would not seem improbable that the amount to be pumped would equal the amount which would seep under the dam designed by the writer. Mr. Morison.

The one feature of the work described in this paper, with which the writer feels least satisfied, is the temporary dam, which, as he has stated, was designed principally for purposes of estimate. The upper rock-fill dam is estimated to contain 450 000 cu. yds., while a similar dam finishing at Elevation 75 would contain 300 000 cu. yds. A like dam, finishing at Elevation 62, would contain less than 150 000 cu. yds., and could be built without difficulty in a single dry season. If the upper portion of the excavation for the locks is completed with the sill at Elevation 47, about 25 000 cu. ft. per second could be passed through the lock excavation, with the water in the lake at Elevation 60. The chances are that this discharge through the locks together with the seepage through the open rock dam would pass the floods of a single high-water season, but an exceptional flood would prove destructive. Moreover, while a crib dam would probably go to pieces in two or three years from decay of the timber, it would be perfectly safe for a year, and a crib dam with an average thickness of 75 ft., finishing at Elevation 65, would contain less than 100 000 cu. yds. Such a dam could be built in a single season, and could be so constructed that it would not be seriously injured by a flood passing over it. Such a dam, which would ultimately be buried in the rock-fill dam, would reduce the volume of the latter within such limits that the remaining work necessary to send the floods safely through the Gigante Spillway could be completed in a single dry season. The writer believes that a careful study will suggest some practical method of dispensing with the temporary dam, and thereby reducing the cost of the dam as designed by him.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 925.

IS IT UNPROFESSIONAL FOR AN ENGINEER TO BE A PATENTEE?*

By Archibald R. Eldridge, M. Am. Soc. C. E.

With Discussion by Messrs. S. Whinery, James Owen, W. C. Hammond and Archibald R. Eldridge.

It may be, that, when drilling for water, oil is struck instead. Thus it was when the writer was reading the discussion on the paper† by Leonard Metcalf, Assoc. M. Am. Soc. C. E., he came upon Mr. James Owen's vehement denouncement of an engineer taking out a patent upon any meritorious product of his brain and energy, and, for the first time in his limited professional career, had "the law laid down" to him, that such was a gross violation of professional ethics. Thinking that others in the profession may be as benighted as he upon what might be, to some, a most momentous question, is his apology for inviting a discussion of this subject.

The writer would state, at the outset, that he has never applied for a patent upon any article whatsoever, not even for a washing machine or a car coupler, the two articles which keep the Patent Office working overtime; and will further state that he has no patent-

* Presented at the meeting of April 16th, 1902.

† "The Antecedents of the Septic Tank," *Transactions*, Am. Soc. C. E., Vol. xlvi, p. 456.

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able schemes in sight at the present writing. His presentation of the subject, therefore, is from an unbiased and unprejudiced point of view, and whatever opinions he may advance are entirely impersonal.

Some one once said, and most truly, too, that an engineer's capital consists of his brains. His brains make up his stock in trade; his brains, or rather their products, he sells. Others may sell the products of their mills and factories; the engineer sells the products of his trained intellect—no other goods has he to place upon the marts of trade. Wherefore should he then be debarred from selling those products to the best advantage to himself, provided always that they are sold "openly and above board" and not "sneaked" into a piece of work or a contract? By the latter terms the writer means that it would be manifestly wrong for an engineer in authority to specify absolutely the use of certain devices or articles of which he was patentee and owner. Let his employer do that, if the articles or devices in question are so meritorious that they deserve exclusive recognition. It might also be questionable for an engineer to patent a device worked out while in the employ of a client; for it may be, as Mr. Owen says, that in that case, the invention would belong to the client and not to the engineer. That, however, seems to be more a point of law than of ethics, and is not germane to the present discussion.

The point at issue, then, simmers down to the question: Can an engineer, working on his own lines, patent any device, which he may discover or invent, without a violation of the ethics of the engineering profession?

Common sense answers "Yes," unreservedly, but Ethics may answer, just as emphatically, "No."

Let it be supposed, and the supposition is within the bounds of both possibility and probability, that an eminent engineer, one of the leading lights of the profession, should, by chance, stumble upon a decided improvement in egg-beaters. Would he debase the ethics of his profession by taking out a patent on his discovery or invention? The writer hardly thinks that the engineering world would decry such an action; why, then, should it object to his taking out a patent on a new and improved method of rolling steel rails, on an improvement in the valve gear of an engine, or on the construction of a freight car. There seems to be no reason, except intangible "Ethics," and

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ethics, like charity, may be stretched indefinitely to cover a multitude of things, even so far as to become a veritable bogie man and a hamper to the "younger" members of the profession; and, in passing, it may, perhaps, be worth while to note that to some of the "younger members" may be given the credit for some of the most valuable discoveries of the engineering world. For example, W. J. M. Rankine is considered, the writer believes, quite an authority on more than one engineering subject, yet even at the time of his death he was little more than a "young" member of the profession.

To look at the matter in another light: Perhaps for an engineer to patent an invention shows a spirit too "money-grasping," instead of working for the good of mankind. That is a pretty theory, but it will hardly hold water when put alongside of the bills rendered for expert engineering opinion and advice. Do not think from this that the writer is an advocate of "cheap" work. Like the rest of his professional brothers, he is not working for his health, and he believes that an engineer should receive full compensation for any knowledge that he may have to sell. Whether the engineer can pass an opinion at a moment's notice, or whether it will take a month of patient research to reach the same conclusion, makes no difference to the client. The opinion is what he wants; and if his engineer is so well posted that he can give it at once, it is simply so much the better for the engineer. He has the goods, he can deliver them promptly, and should be paid accordingly.

The writer hardly thinks it will be contended that an engineer is not in business for all the money he can make in an honorable and legitimate manner. Such a contention would be foolish in the extreme, with a few very rare exceptions; and he also feels assured that every engineer has a contempt for the quacks and "cut-throats" who are in this, as in every other profession; and further, that all engineers are willing and anxious to see their brethren get just and full recompense for their services, so that, in spite of all that may be said to the contrary, engineers are simply business men, and, as such, are entitled to full compensation for their wares, the products of their brains.

Again, the writer does not see or hear of any objection being raised when an engineer copyrights a book which he writes or even compiles, yet, wherein, pray, is to be drawn the fine distinction be-

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tween a patent and a copyright? An article, a device, a method of manufacture may be "patented," whereas a book may be "copyrighted." In either case others than the owners of the patent or copyright are restrained from using the article or the contents of the book without compensation, in one form or another, being paid to the holder of the patent or the copyright. Patent! Copyright! One all wrong; one all right! Rather a case of Tweedledum and Tweedledee, isn't it?

The writer would not have to go very far to find one, or more, engineers, leading ones in the profession, too, who are directly interested in patented articles, and they are not "younger members" of the profession, either. Furthermore, the writer does not believe that they have in the least violated the ethics of the profession of which they are to-day ornaments.

It is virtually impossible to name a book or treatise, written by an engineer, which is not protected by copyright.

In conclusion, it may be stated that the very words condemning the taking out of patents by engineers, the very sentences which caused the writing of this article, are, themselves, "patented," and to reproduce them the publisher of the reproduction must pay, not in money it is true, but by printing also the full title of the paper from which they are taken, the name of the author, etc., according to the following, which is found on the title page of the "*Proceedings* of the American Society of Civil Engineers," to wit:

"Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author, page reference, and date of presentation to the Society, are given."

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DISCUSSION.

Mr. Whinery. S. WHINERY, M. Am. Soc. C. E.—The question propounded in the title of this paper is one upon which there exists a radical difference of opinion in the engineering profession, particularly among civil engineers. It is a question of importance, from more than one point of view, and, therefore, it merits careful consideration.

It is desirable that a rational conclusion should be reached and generally accepted by the profession and the public. The paper, therefore, is timely and pertinent, and should be discussed fully and candidly.

It will hardly be claimed, except by those who argue that the whole theory of patents is wrong, that the engineer is debarred by either law, morality or professional ethics from taking out a patent under any circumstances. No valid reason can exist for denying him the right to patent inventions that have no conceivable connection with, or no relation to, his profession.

Using an illustration referred to by the author: If an engineer invents a novel egg-beater, he may, if he chooses, apply for and accept a patent on his invention, and dispose of it for a consideration, without violating accepted moral principles, or transgressing the strictest canons of professional ethics. This much may be accepted without discussion. The only question, then, at issue is, may the engineer patent inventions which relate directly or indirectly to the profession, or to his professional work, without violating either the law, the accepted moral code, or the ethics of the profession?

It will be conceded that the public at large neither has nor can have any peculiar legal or moral claim upon a man because of his vocation or profession; every citizen owes certain obligations to his fellow-men and to organized society, and he may pay a part of these obligations in the particular kind of current funds that he may be best able to supply. Thus the physician may contribute to the general good his assistance and advice on questions of public health; the lawyer, his help in framing public laws, and the engineer may contribute so much of his special knowledge as the public is entitled to, in designing schemes for public improvement. But how much is the public entitled to? Somewhere there must exist a line beyond which contributions to the public good must become benevolence, rather than duty. The public has a right to demand that which is duty, but not that which is benevolence. In the way of monetary or property contribution, the duty of the citizen is discharged when he pays his taxes, and the public has no right to demand or ask more.

There are but few who deny that inventions are personal property—the property of the inventor—and a patent is merely a certificate of ownership issued by the authority of the government. The public

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has no more claim upon the personal property of the inventor than upon the chattels or lands of other citizens; therefore, we may exclude the general public from this discussion. There are, then, left to be considered, the rights of but three parties or interests: The profession, the client of the inventing engineer, and the engineer himself. Mr. Whinery

What claim has the profession upon the inventions of its individual members?

It cannot be held that the profession has any legal right to the personal property of its members; nor can it be claimed that the member is bound by any moral obligation to contribute his property to the profession. But, outside the pale of the law and the moral code, it is recognized that the profession has claims upon its members which are not less real because they are somewhat difficult to define. They are embraced in the general term "Professional Ethics."

In the first place, the member has a material personal interest in supporting and advancing the character and standing of the profession before the public, because his own standing in the eyes of the public is affected thereby. From the material point of view, the higher the services of the civil engineer are rated, the more remunerative will be the work of the individual engineer.

Another and a higher motive is found in the fact that the character and dignity of the profession is, to a certain extent, reflected upon its individual members. Personal interest, therefore, dictates that the individual shall render such services as he may be able to the elevation of the profession at large. A higher and still more worthy motive impels the individual to feel a pride in and be loyal to the highest interests of the profession, for the good of the profession itself, and to extend to his fellow-members the most refined courtesy and loyal assistance in all proper ways. This may be accepted as a sound general statement, but difficulty arises when we attempt to define just how far an engineer is called upon to go, in contributing from his personal property, whether material or intellectual, to the profession or to its individual members.

We may reasonably expect that he will contribute his individual knowledge to the general stock, though he may expect, and we may not debar him from, some professional compensation therefor.

When a valuable paper is read before this Society, the profession and its members are benefited, but the author may properly expect some compensation in the way of increased reputation. If an able book on some branch of engineering is written and published the author benefits the profession, and deserves our warmest thanks, but we do not claim that he should not receive a suitable royalty from the publisher. Other illustrations might be given, but they are unnecessary.

It must be accepted that in the profession, as elsewhere, "the laborer is worthy of his hire," and therefore entitled to it. We may

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Mr. Whinery. assume that the products of an engineer's brain are as much his personal property as are the products of his hands, and that he is free to make such disposition of both kinds of property, subject to all proper and usual limitations, as in his judgment may be to his best interests.

Inventions relating to or connected with the engineering profession may be divided into two general classes.

First, those that relate to what we may call the engineer's workshop, such as special instruments or labor-saving appliances, for doing work, the scope of which is confined to the personal use of engineers. With regard to these, it may be an open question whether loyalty to the profession, and courtesy to its members does not require that their use be made free to all. In the medical profession it is distinctly held that it is unprofessional to patent inventions of this class. This position would be strengthened if it could be shown that the profession would reap the whole advantage of the inventor's generosity. Unfortunately, this is not always the case. For instance, an engineer may invent a valuable labor-saving instrument. It is made and sold by an instrument maker, who fixes the price, not at cost and a reasonable profit, but at what he thinks he can get, independent of any royalty the inventor might exact. The result would be increased profit to the maker or dealer, without any benefit to either the profession or the inventor. Upon the whole, however, the position of the medical profession seems reasonable and proper, and the speaker would favor its adoption by engineers.

To the second class belong those inventions that confer no special benefit upon the profession or its members, but which accrue to the benefit of clients, manufacturers, contractors, or the public at large. As a pertinent illustration we may cite improved methods or processes for dealing with sewage. The practicing engineer may very properly recommend to his clients such improved processes, whether patented or not, if he is convinced that, all things considered, including royalty, their use will result in benefit or economy to the client. But such recommendations, probably, will not add to or detract from the engineer's professional reputation or advantage, nor will it increase the fee he may charge for his services. Consequently, he receives no benefit, nor does the profession, from the use of the process. If benefit results, it accrues wholly to the client or to the public at large.

The engineer, presumably, will not recommend, and the client or the public will not use, the process, unless they believe that its use will be advantageous or profitable, and, if advantageous or profitable, they cannot reasonably object to compensating the inventor. As well might they claim that because they can buy an inefficient tool for a certain price, the manufacturer of a more efficient and possibly more expensive one must sell it to them at the same price. Now, unless it can be

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shown that an inventor has no property right whatever in his inven- Mr. Whinery.
tion, it seems preposterous to say that he is not entitled to, and may not justly collect, a fair compensation for its use. And since such use is not for the benefit of the profession or its members, it cannot alter the conclusion if the inventor happens to be an engineer. Much more might be said on this branch of the subject, but it seems unnecessary.

The fact that persons calling themselves engineers, whether legitimately or not, may have attempted to foist upon the public so-called inventions that have no real value, or may have claimed the invention and ownership of processes or devices that are not patentable, because of lack of novelty or because of prior invention, and may have harrassed engineers or their clients therewith, is no argument against the soundness of the conclusion at which we have arrived.

There seems to be an impression among many that, in the medical profession, it is considered unprofessional to take out patents of any kind. This is not the case. The ban is placed only upon patents for professional instruments and devices, and upon medicines and methods of treatment. The argument against patents for medicines and methods of treatment is based largely upon the consideration that they are primarily for the benefit of the sick and suffering, and that ordinary benevolence would render it unseemly to prevent their freest use, or to place a tax upon human suffering. No such argument applies to the great majority of industrial inventions, which are the special field of the engineer, and the cases are therefore clearly differentiated.

Coming now to the second class of parties interested, what claim has the client upon the inventions of the engineer?

Two classes of cases must be considered: First, those in which the investigations of the engineer have led to important inventions while in the client's employ, and relating to the client's business; and second, those in which the engineer may find it to the client's advantage to make use of inventions perfected by him before he entered the client's service, and quite independent of the client's business.

In cases of the first class the legal status of the two parties is well settled and clearly understood. It may be stated briefly thus: If A is an employee, say a mechanic in the business or shop of B, and invents an improved device or process relating to the business of B, A may take out a patent covering his invention. B will be entitled to make full and free use of the invention in his own business or shop, but will have no other claim upon it, and A may sell the patent in whole or in part, subject, of course, to B's right of use, as stated previously, to any other persons, not excepting the business competitor of B. Time does not permit taking up the arguments for and against the soundness of this legal dictum. Its universal acceptance by the courts, in

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Mr. Whinery. this country, at least, indicates that it is founded upon right and equity. And as law and morals are practically synonymous, it must be in line with good morals. Unless, therefore, it can be shown that the relation of engineer and client differs from that of employee and employer it must be good law and sound morals for the engineer who may happen to be an inventor; and, as professional ethics is based on the moral law, it must be sound from the ethical point of view.

Cases of the second class are embraced mainly in the first, except that there can be no possible question as to the ownership of the invention. It will scarcely be held that when a client employs an engineer he thereby acquires any right to make use of the engineer's private property, whether that property be his house, or his invention. There is one difference, however, in the two classes of cases which merits notice. The engineer may not recommend the use of his own patented invention to his client without the fullest statement of the facts, including the royalty expected, and even then may not press its use upon his client. Nor may he make use of his invention in work under his charge without the full knowledge or consent of his client, even if no royalty is charged or expected.

The third party in interest is the engineer himself. Upon his relation to the subject we must content ourselves by touching very briefly.

If neither the profession nor his clients are unjustly or injuriously affected by the fact that the engineer is an inventor and patentee, he has the undoubted right to be his own judge as to whether, being an inventor, he shall, or shall not, take the necessary action to place this intellectual property under the protection of the same law that guarantees to him the exclusive use of his chattels and his lands. If he chooses to avail himself of that protection, and to control and manage that property so as to yield him its full value, he may do so without trespassing on any rights of his fellow citizens and without violating any reasonable code of professional ethics.

There can be no wrong in the engineer holding, using, or disposing of property honestly and honorably acquired. Whether that property is the result of physical or mental effort does not matter, and the possession of the one should no more subject the owner to reproach or professional discredit than the possession of the other.

There is another aspect of the matter, which, while it has no relation to morals or to professional ethics, deserves consideration. Assuming that the inventor might be actuated by pure benevolence in working out and perfecting valuable inventions for the sole benefit of the public, his object might be defeated if he declined or neglected to take out a patent.

To develop and introduce an important invention requires usually the expenditure of a large amount of money. The inventor is not

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Mr. Whinery.

often able to supply this money himself. He must seek the aid of capitalists. They will not supply the necessary means unless assured that what they invest will be returned with liberal profit. They must have some assurance that, after they have spent large sums for the development and introduction of the invention, they will be able to prevent competitors from robbing them of the reward for their labor and expenditure. This can only be accomplished through patenting the invention. Without the protection of a patent they would not supply the necessary money, and the invention, however valuable it might be to the public, would not be made available for its use. Without the protection of a patent the inventor of the railway air-brake could not have obtained the capital to develop, introduce and secure its general adoption at so early a period, if at all, and probably, even now, it would not be in general use. That its rapid introduction and use has been of incalculable value to the public, as well as to the railroads, no one will deny. Thus, the patenting of inventions is indirectly beneficial to the public, and if the inventor reaps a liberal share of the benefit, no one can justly complain.

The writer believes that the time has come when the profession should take positive and unmistakable action on this question of the right of the engineer, not only to invent, but to protect his invention in the way the law has provided.

Such action is due to the profession, which should not be placed in the anomalous position of being divided against itself. It is due to the individual engineer, who is entitled to know what the profession regards as professional or unprofessional. It is due to the engineer's clients and to the public, in order that they may know what to expect when dealing with engineers who may be also inventors.

Mr. Owen.

JAMES OWEN, M. Am. Soc. C. E.—This paper is full of good thoughts and ideas upon the subject of the breach of professional ethics occasioned by a civil engineer acquiring a patent, and, while it is really intended as an *ex parte* argument in favor of such a practice, a careful analysis of the author's statements evinces a tendency somewhat contradictory to his assertions.

The great complaint of the civil engineer in the past, and to a more limited degree at present, is the lack of appreciation, by the community at large, of the status and abilities of the professional engineer, and it is easy to realize such a condition when engineers of the author's standing fail to appreciate it for themselves.

The whole point at issue is a proper definition of the word profession, and to determine accurately what is a professional obligation in civil engineering is a very difficult problem. In the legal and medical professions, tradition and legal enactments have fairly crystalized practice, so that the ethical codes of these professions are well defined, and transgressions of such codes are rare, and when made are accompanied with proper and defined penalties.

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Mr. Owen. In the kindred profession to civil engineering, *viz.*, architecture, the subject of ethics has been a matter of consideration for a long time, and the speaker believes that a satisfactory professional status for architects has been defined, but years will elapse before it is fully and unanimously accepted.

Taking the architect as basis for argument, it may be clearly appreciated that any one designing and also constructing a building is not acting in a professional capacity, even though it may take more brains and energy than merely to make the plans and superintend the construction. Applying this creed to engineering, any engineer who departs from a line of practice in which he is delivering to his client more than his own thoughts and ideas is departing from legitimate and professional practice; for these thoughts and ideas must and should be untrammeled by any question of individual interest of the engineer himself, and the resultant, whether delivered by letter or drawings, belongs to his client.

The first problem to be adjudicated in this controversy, and more pertinent probably than any other point, is: Is it of advantage to the engineer to be a professional man, and is it also of advantage to the country at large?

So much of the world's progress of to-day is dependent upon the free thought and careful study by engineers generally that it may be urged that by limiting an engineer's line of action to a comparatively small sphere good results may not ensue, and the pre-eminence achieved by engineers here, under such conditions, may be easily lost.

Looking carefully over the field of civil engineering, with its varying ramifications, it can hardly be said that such a result can be feared, as it shows that, as time goes on, the purely professional practice is gaining greatly on the so-called merchantable practice.

Take bridge construction: In years gone by, the engineer, in most cases, designed and erected the structure. To-day, the best and most costly structures of that class are being designed by purely professional engineers. In pumping machinery, so long considered a purely manufacturing industry, the designs of professional engineers have shown better results in economy and duty. In railroad construction, to-day, the engineering is purely professional; and in water and sewerage works, by far the larger proportion is constructed on professional lines. Thus it will be seen that there will be a very small limitation by the engineer confining himself to purely professional lines, and if such limitation is in any case individually harmful or irksome, there is no prohibition to any man acting in other lines, except that his work will not be professional.

It has been said that the crystalized practice of any engineering field becomes in time a trade, but it may be said also that a systematic trade does not require the services of a professional engineer.

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Given these general outlines of professional engineering, the inference can be drawn that it is professionally wrong for an engineer to acquire an interest in any patent. If he acquires a patent and uses it there is certainly no moral delinquency, but it lowers the dignity of the profession, and lowers its appreciation by the public at large, and it is gratifying to note the public disapproval based on a report of a controversy which has arisen in a large city where a professional engineer has attempted to force the use of a patented article on construction work over which he has control. Mr. Owen.

In conclusion, the speaker desires to endorse almost all the remarks in Mr. Eldridge's paper, and is satisfied that, if their application be made on purely professional lines, there will be no divergency of opinion.

W. C. HAMMATT, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read with considerable interest the discussion as to whether or not it is unprofessional for an engineer to be a patentee; but can add little to the already very complete statement of Mr. Whinery. Mr. Owen seems to have departed from the original question, in order to cite the case of an engineer putting the sale of his patented articles ahead of his professional duty, or, worse yet, using his professional standing to push the patented article for his personal gain. In a case of this nature all will agree with Mr. Owen, and discussion on this point is superfluous. In the other case, however, of an engineer conceiving a device which, be it the result of momentary inspiration, or of long study and research, nevertheless owes its origin to the professional education or experience of the inventor, he would fail to reap the reward due his genius, without benefiting the profession or the public, by not taking out a patent. Mr. Owen states that the taking out and making use of a patent by an engineer "lowers the dignity of the profession, and lowers its appreciation by the public at large." Quite the contrary. An engineer who has patented a successful device, or a number of them, along a certain line of his profession, comes to be considered by his confrères and by the general public as an authority, or "expert," in that line. Mr. Hammatt.

There is nothing in the ethics of the profession prohibiting an engineer from charging his client an adequate fee for work performed or advice given, which is turning his engineering ability to monetary gain. Why is it any worse to collect a small amount, in the form of a royalty, from each of a number of persons benefited by his brain work, than to collect a larger amount from a single client for services performed along the same lines?

ARCHIBALD R. ELDRIDGE, M. Am. Soc. C. E. (by letter).—Mr. Whinery has handled this subject in such an able manner that he leaves little to be said, and Mr. Hammatt, in a few words, has clinched the favorable criticisms made by Mr. Whinery. As Mr. Hammatt Mr. Eldridge.

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Mr. Eldridge. says, Mr. Owen has departed from the original proposition by speaking of an engineer putting the sale of his patented invention ahead of his professional duty, or using his professional position to push the sale, or use, of a patented article in which he is financially interested. The writer thinks that he covered that point in the paper, when he stipulated that an engineer should sell such an article "openly and above board," etc., and considers that he granted that any other proceeding would be extremely unprofessional, and he will go further and say that he feels assured that the entire engineering profession would condemn such practice in no unmeasured terms. Mr. Whinery says that the question at issue is one which should be settled once for all, so that it will not appear that the profession is divided against itself on so important a matter, and the writer feels confident that the question is now settled, and that an engineer can patent any article which he may invent, and that he can dispose of the rights for using the same, but always "openly and above board," without violating the ethics of one of the most honorable professions of the time.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 926.

A BRIEF HISTORY OF ROAD CONDITIONS AND LEGISLATION IN CALIFORNIA.*

By Marsden Manson, M. Am. Soc. C. E.

With Discussion by Messrs. James Owen, George W. Tillson and Marsden Manson.

"The good roads agitation," which forced itself upon the people of the United States a decade or more ago, took definite shape in California in 1895. For several years previous, highways were the subject of earnest consideration by organized official and semi-official bodies and by individuals throughout this state. The most important and influential of these bodies was a road convention, held in Sacramento in 1894, which was inaugurated by the Sacramento Society for the Prevention of Cruelty to Animals, and was attended by delegates from various public and quasi-public organizations, and by individuals.

The result of the discussions throughout the state, and notably of the convention above mentioned, was a law,† approved March 27th, 1895, creating a Bureau of Highways for two years.

This law required the appointment of three commissioners, who were to make a broad and exhaustive study: First, of the laws apper-

* Presented at the meeting of April 2d, 1902.
† Statutes, California, 1895.

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taining to highways in this and other states; second, of the physical features of the state and their relation to a system of roads; third, of the economic and legal status of the highway conditions in each county in the state; and finally, to make a presentation, of the facts and conclusions reached, in the form of a report, commending such measures as the Bureau deemed advisable. The law also required that one or more members of the Bureau, during each of the two years of its existence, should visit each county in the state, and hold public meetings for the consideration and discussion of matters pertaining to highway improvement. To carry out the provisions of this law, an appropriation of $31 000 was made.

The area over which these duties were to extend embraces some 153 600 sq. miles. It confronts the Pacific Ocean along a coast line more than 1 000 miles in length, between the 42d and 32d parallels of north latitude, and lies between the 114th and 124th meridians of west longitude, extending through 9½° of latitude and 10° of longitude. This area, if laid out upon the Atlantic Seaboard, would cover the territory from New York City to Savannah, or nearly the area of one-fourth of the original thirteen states. If laid out upon the European and African Coast, in corresponding latitudes, it would reach from Valladolid, in Spain, across the Mediterranean Sea to Morocco, the capital of the country of the same name, in Africa.

These comparative areas are shown on the accompanying maps, Figs. 1 and 2, which convey an idea of the extent of the country over which the work was to extend.

Configuration.

The area under consideration is principally mountain and rolling land of great diversity of features. Two ranges of mountains traverse it from northwest to southeast—the Sierra Nevada in the eastern, and the Coast Range in the western part of the state. The mean elevation of the crest of the former is about 8 000 ft., and that of the latter about 3 000 ft. Besides the numerous valleys contained between their spurs and lateral ranges, their foot-hills enclose the great central valley of California. The area of this valley is about the same as that of the valley of the Po; the average dimensions of that portion which is below the 400-ft. contour above sea level being 400 miles long and 40 miles wide. Tehachapi Pass, at the extreme southern end, and

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SKETCH MAP, SHOWING THE RELATIVE SIZE AND LATITUDE OF CALIFORNIA COMPARED WITH THE EAST COAST OF THE UNITED STATES.

FIG. 1.

SKETCH MAP, SHOWING THE RELATIVE SIZE AND LATITUDE OF CALIFORNIA COMPARED WITH THE WEST COAST OF EUROPE AND AFRICA.

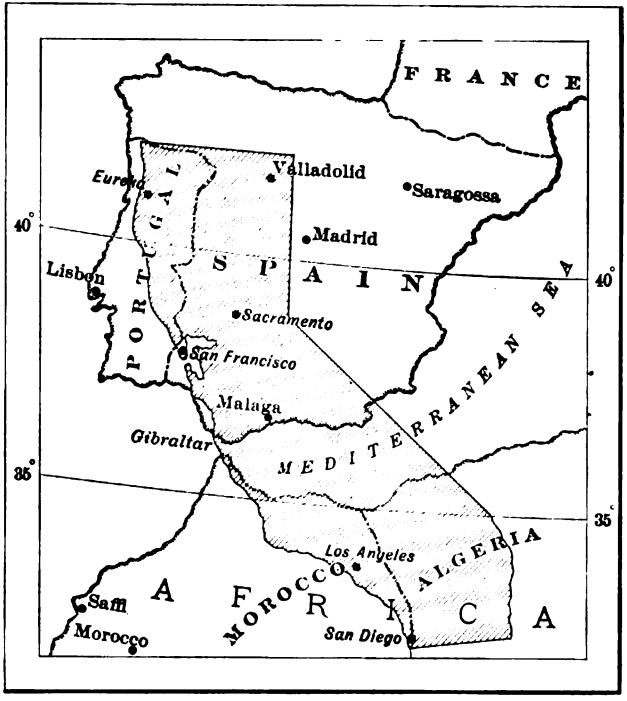

FIG. 2.

Acme Summit, at the base of Mt. Shasta, are important entrances into the great valley. The Sacramento and San Joaquin Rivers, and their tributaries, water and drain the valley; their combined water-sheds embrace an area of 58 100 sq. miles, situated principally upon the western slope of the Sierra Nevada and the eastern slope of the Coast Range. The edges of this water-shed are the crests of the two ranges of mountains, except an area extending into Oregon in the drainage basin of Goose Lake, which lake when full drains into Pitt River, one of the main tributaries of the Sacramento. The eastern crest rises from 6 800 to 15 000 ft., and the western crest rises from a single tide-level gap at Carquinez Straits to elevations of 900 to 9 000 ft. The gaps in these mountain ranges, and the ridge and valley lines subdividing and lying between them, constitute controlling features in the development of a road system.

Climate.

In variety of climate, the area under consideration presents a wider range than do both the comparative areas just described. Besides the climatic variations, due to difference of latitude or distance from the sea, California presents a wide range of temperature, due to differences in elevation. These differences extend from the arctic temperatures of 15 000 ft. above, to the semi-torrid climate of areas 400 ft. below, sea level. Between these extremes every grade of temperature and exposure exists. In the hotter portions, fruits and vegetables grow without intermission during the entire year; while upon the colder heights frosts form in midsummer, and only the hardiest boreal plant life exists. Severe frosts and freezing are limited to the most elevated parts of the state. In the greater portion of the higher mountains snow falls before severe freezing weather occurs, and remains until the warm weather of spring, thus protecting from frost action the road-beds in these regions; and hence no great precautions are necessary to resist its injurious action upon roads.

Rainfall.

California occupies a position between the north temperate and the equatorial rain belts. The northern part of the state is just within reach of the summer rains of the north temperature rain belt; and the mountains of the extreme southern part of the state are frequently

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reached by the "Sonora rains," which extend in summer over Northern Mexico and into Arizona, and which originate in the equatorial rain belt. The winter rains of the entire state are principally dependent upon the southern extension of the north temperate rain belt as that hemisphere is turned away from solar energy. Therefore the state has winter rains, and enjoys a comparatively rainless summer.

The mean annual precipitation in the northwestern part of the state and upon the exposed flanks of the Sierras and Coast Range is 70 to 80 ins.; while in the southeastern portion of the state there are comparatively rainless areas, receiving from 2 to 4 ins. annually. Between these two extremes, again, there lies every degree of annual moisture. This precipitation, however, is rarely so heavy that a properly located, well-surfaced, and well-drained roadbed will be badly washed.

Vegetation.

The distribution and variety of vegetation over an area of such extreme range of climatic and moisture conditions are also of wide range, both in species and density. The forest growth is sometimes so dense that more than 1 000 000 ft. of lumber can be cut from a single acre, but in the drier portions of the state it is difficult or impossible to get shade trees to grow, and other areas are absolutely devoid of vegetation. In the densely timbered areas, clearing and grubbing are expensive, and the shading of the roadway causes it to remain wet longer than is desirable.

Natural Resources.

Besides these physical features which have to be considered, there are four belts of natural resources which have controlled the time, locus, and mode of development of certain industries. These directly influence a road system, and determine controlling points of population and trade.

In progression eastwardly, and in order of development, these are:

First.—The mineral belt, commencing in Del Norte County, and reaching southeastwardly through the entire limits of the state;

Second.—The agricultural belt of 16 000 sq. miles of fertile land, in the Valley of California;

Third.—The timber belt, commencing in the northwesterly corner of the state, and extending southeastwardly to the Golden Gate;

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Fourth.—The fruit belt, commencing at the base of Mt. St. Helena, and extending southerly to the national boundary.

Each of these belts is interspersed with sources of wealth other than those which characterize it; as, for instance, the Sierra timber belt which lies east of and along the mineral deposits on the flanks of the Sierras, and the foot-hill fruit belt at their base. The foregoing classification, however, marks the predominant natural wealth of each of the various sections.

At advantageous points along each of these belts, centers of population are growing up. The problem of laying out the lines of communication which must for all time control the travel and traffic of the state, therefore, must be subservient to two great controlling factors: First, these roads must lie along those lines, determined by Nature, which offer the best grades and alignments, second, they must subserve the economic purposes demanded in the development of the state.

Occurrence of Road-Building Materials.

The Department of Highways extended the work of investigating the various materials suitable for road surfacing, and these investigations showed that there is an abundance of materials over the entire state which can be utilized with great benefit to economic road maintenance. In almost every county adequate materials exist, which, if utilized with skill and judgment, will make good metal. The failures to utilize these materials have been due in part to a lack of knowledge as to their character, and of technical skill as to the mode of using them.

All hard, silicious rocks, such as chert, jasper, quartz, quartzite, etc., the volcanic rocks (trap and basalt), and a wide series of metamorphic rocks occur abundantly. In addition, bowlders, cobbles and gravel of miscellaneous composition, but exceedingly hard and durable, are scattered over great areas. Hard limestone and some of the silicious shales occur in many counties. Occurring with these rocks are others which are inferior or useless for road surfacing. Many instances of failure in the use of rock have been due to the selection of the wrong material. Again, the proper selection has been made, but the materials have not been properly utilized. The best rock roughly cracked up and dumped over a road will no more make a road surface than shingles dumped on a shed and scattered with a rake will make a roof.

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The best rock must be selected, properly crushed and screened, and then spread, sprinkled, and rolled in layers over a well-graded and drained surface. In one county, an excellent binding material, a soft limestone, was found. It was being hauled a considerable distance, and used almost alongside of the hardest and most durable chert, with which a small proportion of the limestone should have been used as a binding or cementing material. This would have given a better and more durable road surface and saved the long haul from the limestone quarry of nine-tenths of the stone used.

Legal Considerations.

In addition to the physical conditions, there are others imposed by the administration of the law in the various political divisions of the state. The state is divided into fifty-seven counties, which range in area from the 20 000 sq. miles of San Bernardino County to the 43 sq. miles of San Francisco County; and in taxable wealth from the $400 000 000 of this latter county to the $260 000 of Alpine County. Furthermore, these counties are divided by law into fifty-three "classes," in each of which material variations are presented in the number, compensation and duties of county officers.

The entire matter of road-building and maintenance has been in the hands of the Boards of Supervisors of these counties, or in the charge of Road Overseers, elected in the road districts into which the counties were subdivided. The abuses incident to the latter control were so great that the roads were placed under the control of the Boards of Supervisors, each member being made Road Commissioner of his district. The control of road funds under Road Overseers became so vital a matter in local politics that at elections the votes for Presidential Elector were sometimes "swapped" for those for Road Overseer.

The laws under which the county officers administer road affairs have been so modified, amended and added to, in the past forty or more years, that they are contradictory and inapplicable. Amendments made to correct difficulties, abuses or even physical conditions, in one portion of the state have made "confusion worse confounded" in other portions. Hence, the exact provisions of the code were not only unascertainable, but in many instances inapplicable, and, furthermore, subject to different interpretations by different officials.

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The obstacles to be dealt with are therefore of two classes: (1) physical; (2) political.

Of these, the first can be met by the engineer in almost any form, provided the necessities for overcoming them warrant the raising of the required funds.

The second class, or political obstacles, are so entrenched by custom and control that only long devotion to civic duty by a majority of the voters of a county or state can correct them, and the attack cannot be made successfully until justified by the state of public opinion.

The system of placing county road matters in the hands of a Board of Supervisors, not one of whom is trained for the work, is radically wrong. It happens frequently that members of these boards are faithful, energetic men, who are devoted to the duties of the office, and discharge their obligations to the public with honest efficiency. Many instances of this kind were met by the writer in various parts of the state.

The works of these officials might be considered models, when the obstacles and difficulties under which they were executed were considered. But, unfortunately, this was not the rule. In the majority of instances road moneys were regarded as the funds from which to pay political debts and obligations, or were to be called upon just before elections for the purpose of putting an army of "workers" on the roads where they "would do the most good," not to the roads, but to some candidate.

This absurd and obscure condition of the law and its administration can be understood only when the actual conditions of road management in the entire country, previous to and during that period of its history in which California has developed, are considered; and, without the consideration of these conditions, road mismanagement by an intelligent people is inexplicable. Hence it becomes necessary to review very briefly the modes of road development in the United States since the advent of the railroad.

Historical Notes.

In the early part of the century, the prosperity and development succeeding the War of the Revolution required ample and extended means of communication. The common roads of the

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country were supplemented by toll roads, which were built by companies, on franchises granted for variable periods. These toll roads were frequently built of plank, but were generally of earth and gravel.

The exaction of toll upon these roads became burdensome and restrictive of development; consequently, the acquisition of many of them became a public necessity. From that date to the present we have had it impressed upon us that whenever those utilities which the people should provide and control for themselves have been farmed out to corporations, oppression has resulted—no matter whether the farmed-out franchises were a toll road from Pittsburg to Philadelphia, or a transcontinental railroad to the Pacific Ocean.

The necessities for cheap transportation became so great that the United States Congress, cabinet officers, the President—in fact, almost all the functions of the National, State and County Governments were engaged in the study and practical application of the science of road building. Reports of cabinet officers, messages of the Presidents, and the views of the leading statesmen of our country, bearing on this subject, are very common in Government documents during the early decades of the nineteenth century. Notable among these are:

1.—The Act admitting Ohio, in 1802, and setting apart 5% of the sales of public lands as a road construction fund. These sales netted something over $750 000 in a few years.

2.—The bill introduced by Mr. Tracy, of Connecticut, and approved in 1806 by President Jefferson. The debates on this Act show how deeply the road question interested the country.

3.—The speech of Mr. Calhoun in favor of placing the bonus on all bonds sold, and dividends of the national banks, in a fund for the construction of roads and canals; this fund would have amounted to nearly $750 000 annually. This bill, however, was vetoed by President Monroe, in 1817, on the ground that it was unconstitutional, even if the States gave their consent to the provisions of the law.

An extensive system of roads was projected and partly constructed by Congress. The most noted were the "National Turnpikes" in Maryland, Virginia, Pennsylvania and Ohio, which aggregated several

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hundred miles in length, and received Congressional appropriations amounting to $7 000 000 in annual appropriations of from $30 000 to $500 000.

Finally, in 1834–35, an appropriation of $300 000 was made for repairing the National roads in Pennsylvania, Maryland and Virginia, and when repaired they were transferred to these states.

Just as road building, fostered by our government and placed in the hands of skilful engineers, became a science, the invention of the locomotive and the consequent development of railroad building came about. At once, all the energies of our people were centered upon this new mode of transportation. So great was the impetus thus given, that the United States soon ran far ahead of other countries in mileage of railroad. For generations the building of roads was forgotten, and the building of railroads stimulated and fostered. The highest skill of the trained engineers of the country has been and is engaged in the general and special work of building, equipping and operating railroads.

During this period of railroad growth and development, roads have been neglected to such an extent that generations have grown up who have never seen a road. They have learned to speak of streaks of dust or mud, as the case may be, as roads—have actually learned to regard them as such, and solemnly dedicate the same to public use, with all due legal form, as if they really were well-located, graded, drained and thoroughly metaled highways. The engineers of the country have so seldom, until lately, been engaged on roadwork that they, too, have looked upon road building as a lost art which at one time was practiced by the ancients.

Under these conditions California was settled, and her laws for road location, construction and management, were framed and developed during a period when road building was at its lowest ebb.

The road system of California, therefore, grew up during a period of general road decadence throughout the entire United States. It has also had a makeshift system grafted upon it from the start. The energies which led the pioneers to traverse a continent in the search for gold was coupled with a love of home which limited their contemplated stay in California to the shortest possible period consistent with the acquirement of wealth. They were, therefore, content with trails, roads and bridges which were essentially temporary or make-

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shift in character. Consequently, Californians have learned to regard road building and maintenance as the result of temporary expedients resorted to only when forced by necessity. In some instances, where wise forethought has prompted permanent work and materials, the step has been regarded as an expensive "experiment," and the officials inaugurating these improvements have been accused of extravagance.

It is, therefore, not so much a matter of surprise that the condition of affairs mentioned in the earlier part of this paper grew up along such systemless and extravagant lines.

But the expense of operating bad roads forced a consideration of the methods necessary for correct road location, construction and maintenance. The state had in 1895 reached the limit of development possible under bad roads, and further development was and is yet dependent upon the systematic construction of highways of the most approved type.

Under these necessities, the law of 1895, previously mentioned, was passed.

Legislation.

After two years spent in investigating the subject and in examining the conditions thoughout the state, the Commissioners of the Bureau of Highways recommended minor changes in the then existing laws—principally to harmonize them—and four general laws. These were:

(1)—An Act to classify the roads of the State into (*a*) State Highways; (*b*) County Thoroughfares; (*c*) District Roads.

(2)—An Act empowering the State to gradually take charge of, construct and maintain highways of the first class, and making provision therefor, by the levy of a tax of 2½ cents on the $100 of assessed valuation.

(3)—An Act regulating the width of tires.

(4)—An Act creating a Department of Highways.

(5)—An Amendment reducing the maximum rate of road taxation from 40 to 35 cents per $100 of assessed valuation.

The objects and principles of these laws had been announced and discussed, during the year preceding their introduction, in every county seat and important town, and had met with the general approval of the citizens and the press. Each of the political parties

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had put a "plank" in its "platform," looking to the betterment of road conditions, which was supposed to pledge the members of the Legislature to remedial measures.

Of these proposed acts, only one—that regulating the width of tires—became a law. This was drawn to go into effect in 1900. It met with the disapproval of the dealers in and manufacturers of wagons, and at the extra session in 1899–1900 was rendered inoperative.

The law classifying roads was radically changed by the introduction of clauses which would permit of making unimportant roads State Highways, when they could never form the essential parts or links of a great highway system, and would, furthermore, place no limit on the mileage of such additions to the State's obligation. Recognizing the evils of these changes, the Executive withheld his signature.

The bill providing for State construction and maintenance of highways of the first class was amended to entirely alter its purport, and was vetoed by the Governor.

The recommendation to levy 2½ cents on the $100, for the construction of State Highways, was also radically changed by extracting essential features and incorporating them in another bill, practically placing 85% of that levy to the credit of the counties, to be expended under and by county authorities; also, providing that state officials should indorse bonds issued by the counties, and, in default of the counties paying the interest thereon, that the State should collect and pay such interest. These provisions were contrary to the plain provisions of Article IV, Section 31, of the Constitution, which prohibits the granting of the money or credit of the State to political or other corporations or to persons. These measures failed to become laws, as did the bill previously mentioned.

The bill creating the Department of Highways was amended, for partisan purposes, so as to give patronage in State elections.

The recommendation to reduce the maximum rate of taxation for road purposes from 40 to 35 cents per $100 was not considered by the Legislature, and, instead of a reduction, the maximum was raised to 62½ cents per $100 of assessed valuation, by the addition of certain district levies.

An act, constituting Chapter CCLIXIV of the Statutes and Amendments to the Code, 1897, was introduced as an "urgent necessity," and

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passed during the last eighteen hours of the session. It was known as the "Clark Road Law." It was never printed or submitted to consideration, and its existence was kept secret from the members of the Bureau of Highways. This law subsequently came before the Supreme Court,* and was declared illegal.

An act intended to provide a connecting link between the roads on either side of the Sierras and between the Yosemite Valley and the basin of Mono Lake was also passed, but, the title being defective, it failed to receive the signature of the Governor.

The Legislature of 1897 also made a conditional appropriation of dimension stone for substructures and of crushed rock for macadamizing the road from Sacramento to Folsom, a distance of about 20 miles. Near this latter point one of the state penitentiaries is situated, and convict labor was to quarry, dress, or crush the rock. The condition of this appropriation of materials was that the county should provide the funds necessary to grade the roadbed, distribute and roll the crushed rock, etc.

This road was to be a model for the state and county officers, who meet biennially at the Capitol.

The work was placed in the hands of a non-technical commission. The members of the Department of Highways undertook to present the matter to the county officials and the people of the county for a bond election. This election resulted very favorably, but the legal provisions and steps were not fully in accordance with the law, and the courts declared the bond issue invalid.

This, in brief, is the history of the first two years' attempt to institute system and economy in road work in California.

The work, however, was continued through a Department of Highways, which took the place of the previous Bureau, and for two years consisted of three members, and thereafter of one member, who was to be appointed for four years.

The Department of Highways thus created continued the work provided for by law, but, by lack of funds, was not permitted to execute any important road work. The members acted in an advisory capacity to County Boards of Supervisors and to County Surveyors. In a few instances opportunities were found to practically advance road building.

* In Davis *vs.* Whidden, 117 Cal., 618.

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In laying out a system of highways it must be recognized that there are two main principles to be followed. First: The main highways of the state and its counties are lines along which the travel and traffic of an indefinite future must move, and it is therefore necessary to locate these lines on the most advantageous ground, irrespective of temporary private interests, which at most can last for only one or two generations. After roads have been located upon these lines no false economy or makeshift methods should be allowed; the drains, culverts and bridges should be of masonry, the roadbed graded to true lines, and a sprinkling plant provided. Finally, the surface should be metaled with the best available rock. To attempt this upon all roads, with the entire road fund of each county, is not practicable. Existing roads, however bad, must be maintained so as to permit of reasonable use, even if this use temporarily entails heavy loss.

The second principle is that true economy shall characterize the expenditure of available funds. The existing roads must be kept in that state of repair which will enable them to meet the requirements of travel and transportation, and, in addition, a gradual change to proper locations, and the substitution of permanent structures for makeshift and perishable ones, must be made.

It was recommended, therefore, that one-half of the road funds of each county be set aside as a "Good Roads Fund," to be used as the necessities of the county might dictate. With the remaining half of the road funds, it is entirely possible to maintain existing roads in as good condition as they have been for some years past. Particularly is this possible, if wasteful methods and political favoritism be set aside, and system and economy instituted. Should this measure reduce the funds below the requirements of special local interests, the Board of Supervisors has it in its power to levy for this purpose the special tax provided in Section 38, Chapter CCLXXVII of the Statutes and Amendments of 1897. The setting aside of a portion of these funds for permanent work was intended as a means of bringing about system and economy. This, however, cannot be accomplished by law alone. It is imperatively necessary that there should be integrity, technical intelligence, and energy on the part of the officials charged with the disbursement of road funds.

The construction and maintenance of bridges was found to be particularly costly in all except two counties. In one the cost had been

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reduced by placing the entire matter of bridge building in the hands of the County Surveyor, an engineer of high skill and integrity. In the other it was realized that masonry was far cheaper than steel, iron and wood. The results of these two examples were freely used by the Highway Commissioners, and the lessons were not wholly lost.

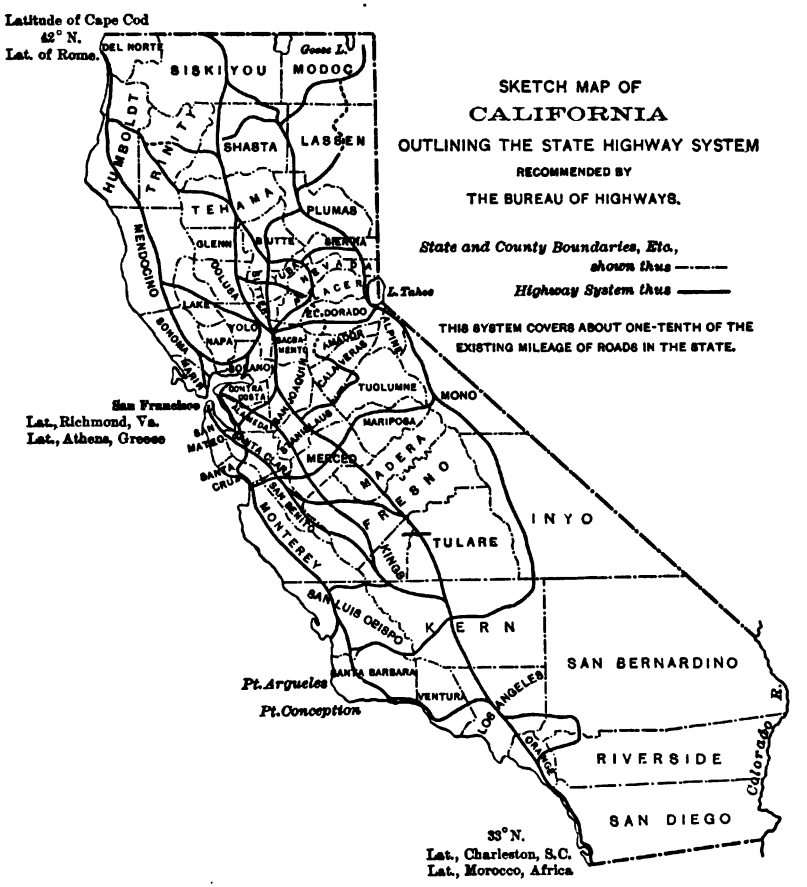

FIG. 3.

The prime reasons why bridge building and maintenance are so costly are: First, in many instances the bridges are expensive because the county officials do not know the actual cost of materials and labor; and, secondly, a lack of technical knowledge in the matter permits the use of the wrong materials.

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Bridges, which should be masonry, are built of very perishable timber, the constant renewals of which exhaust the road funds.

The raising of about $2 000 000 per year for highway purposes is equivalent to a tax levy of 20 cents per $100 on $1 000 000 000. This amounts to $80 000 000 in forty years, which sum, intelligently and honestly expended, would give California a magnificent system of highways; but it requires the introduction of system, skill, and integrity in every detail, and, if the present system be continued, the full benefit of the expenditure of this vast sum will fall short of being realized.

At the next session of the Legislature, in January, 1899, the Department of Highways renewed the recommendations previously made, and added an important amendment, looking to the creation of a good roads fund in every county, but leaving its control under county authorities. This control, as previously outlined, is considered a very essential part of the political patronage of members of the several Boards of Supervisors, and they generally regard any attempt to put road matters on better lines as an interference with their prerogatives and rights. This amendment apportions 50% of all road moneys for the construction of permanent work in a specified order. Considering that the gross annual expenditures by counties aggregate about $2 000 000, this "good roads fund," in a few decades of systematic work, would accomplish much good.

These laws and amendments are appended herein in full. They are not intended to be radical or to subvert the method of road administration, but are designed to introduce system and economy, and to correct gradually the abuses and lack of business methods which had grown to be a part of road administration.

An Act to Classify the Roads in the State of California, and to Define each Class.

The People of the State of California, represented by Senate and Assembly, do enact as follows:

Section 1.—The roads within the limits of the State of California shall hereafter be classified as follows:

The first class to include all highways designated as State Highways; the second to include all highways designated as County Thoroughfares; and the third class to include all highways designated as District Roads.

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Section 2.—The following roads, when definitely located by the Department of Highways, are hereby declared to belong to the first class.

1. A highway commencing on the State line between the State of California and the State of Oregon, at or near the point where the said State line is intersected by the road from Yreka, California, to Ashland, Oregon, and extending thence southerly, along the best grades and alignments, through the Counties of Siskiyou, Shasta, Tehama, Butte, Yuba, Sutter, Sacramento, San Joaquin, Stanislaus, Merced, Madera, Fresno, Tulare, Los Angeles, Orange and San Diego to Tia Juana, in the last-named county.

2. A highway commencing at Crescent City, in Del Norte County, and extending south and southeasterly, on the best grades and alignments, through the Counties of Del Norte, Humboldt, Mendocino, Sonomo, and Marin, to the City of Sausalito.

3. A highway commencing in the City and County of San Francisco and extending thence southeasterly, on the best grades and alignments, through the Counties of San Mateo, Santa Clara, San Benito, Monterey, San Luis Obispo, Santa Barbara, Ventura and Los Angeles to the City of Los Angeles.

4. A highway commencing at a point upon the State Highway through Tehama County, at or near the station of Tehama, and extending thence southerly, on the best grades and alignments, through the Counties of Tehama, Glenn, Colusa, Yolo and Solano, to the City of Vallejo.

5. A highway commencing at the City of Martinez, and extending thence southerly, on the best grades and alignments, through the Counties of Contra Costa, Alameda, San Joaquin, Stanislaus, Merced, Fresno, Kings and Kern to a point on the State Highway at or near the City of Bakersfield.

6. A highway commencing at a point upon the State Highway through Siskiyou County, near the westerly base of Mount Shasta, and extending thence southerly, on the best grades and alignments, through the Counties of Siskiyou, Shasta, Lassen, Plumas, Sierra, Nevada, Placer, Eldorado, Alpine, Mono, Inyo, and Kern to Indian Wells, in the last-named county.

7. A highway commencing at the City of Arcata, in Humboldt County, and extending thence southeasterly, on the best grades and alignments, through the Counties of Humboldt, Trinity and Tehama, to a point on the State Highway through Tehama County, at or near the City of Red Bluff.

8. A highway commencing at a point on the State Highway through Shasta County, north of the City of Redding, and extending thence northeasterly, on the best grades and alignments, through the Counties of Shasta, Lassen and Modoc, to Fort Bidwell, in the last-named county.

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9. A highway commencing at the City of Marysville, and extending thence northerly and northeasterly, on the best grades and alignments, through the Counties of Yuba, Butte, Plumas and Lassen, to Susanville, in the last-named county.

10. A highway commencing at the City of Ukiah, and extending thence southerly, on the best grades and alignments, through the Counties of Mendocino, Lake and Yolo, to the City of Sacramento, thence easterly, through the Counties of Sacramento and Eldorado, to a point on the State line between the State of California and the State of Nevada, at or near its intersection by the Lake Tahoe Wagon Road.

11. A highway commencing at or near the City of Santa Rosa, and extending thence southeasterly, on the best grades and alignments, through the Counties of Sonomo, Napa and Solano, to Suisun City, in the last-named county.

12. A highway commencing at a point on the State Highway running north from Sacramento, and extending thence northeasterly, on the best grades and alignments, through the Counties of Sacramento, Placer, Nevada and Sierra, to a point on the State Highway through Sierra County, near Sierraville, in the last-named county.

13. A highway commencing at the City of Oakland, and extending thence easterly, on the best grades and alignments, through the Counties of Alameda and San Joaquin, to a point on the State Highway through San Joaquin County, south of the City of Stockton.

14. A highway commencing at the City of Oakland, and running thence northerly and easterly, on the best grades and alignments, through the Counties of Alameda and Contra Costa, to the City of Martinez.

15. A highway commencing at Ione, Amador County, and extending thence easterly, on the best grades and alignments, through Amador, Calaveras, Tuolumne and Mariposa Counties, to the county seat of the last-named county.

16. A highway commencing at a point on the State Highway through Santa Clara County, at or near the City of Gilroy, and extending thence northeasterly, on the best grades and alignments, through the Counties of Santa Clara, San Benito, Merced, Mariposa, Tuolumne and Mono, to a point on the State Highway through the last-named county, near Mono Lake.

17. A highway commencing at a point on the State Highway through Santa Clara County, at or near the City of Gilroy, and extending thence northeasterly and easterly, on the best grades and alignments, through the Counties of Santa Clara, San Benito and Fresno, to the City of Fresno.

18. A highway commening at the City of Modesto and extending thence northeasterly, on the best grades and alignments, through the Counties of Stanislaus and Tuolumne, to the City of Sonora.

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19. A highway commencing at or near the City of Hollister, and extending thence southeasterly, on the best grades and alignments, through the Counties of San Benito and Fresno to a point on the westerly State Highway through the last-named county, near Huron.

20. A highway commencing at a point on the State Highway through San Luis Obispo County, at or near San Miguel, and extending thence easterly, on the best grades and alignments, through the Counties of San Luis Obispo and Kern, to a point on the westerly State Highway in the last-named county.

21. A highway commencing at Port Hartford, in San Luis Obispo County, and extending thence southeasterly and northeasterly, on the best grades and alignments, through the Counties of San Luis Obispo, Santa Barbara and Kern to Indian Wells, in the last-named county.

22. A highway commencing at the City of Los Angeles and extending thence easterly, on the best grades and alignments, through the Counties of Los Angeles and San Bernardino, to the City of Bernardino; thence southwesterly, on the best grades and alignments, through the Counties of San Bernardino, Riverside and Orange, to the City of Santa Ana, in Orange County.

23. A highway commencing at a point on the State Highway through Shasta County, north of the City of Redding, and extending thence northwesterly, on the best grades and alignments, through the Counties of Shasta and Trinity, to Weaverville, in the last-named county.

24. A highway commencing at Colusa, and extending thence westerly, on the best grades and alignments, through the County of Colusa, to a point on the State Highway through said county.

25. A highway commencing at Markleeville, and extending thence easterly, on the best grades and alignments, to a point on the State Highway through Alpine County.

26. A highway commencing at Mariposa, and extending thence northwesterly, on the best grades and alignments, to a point on the State Highway through Mariposa County.

27. A highway commencing at the City of Visalia and extending thence westerly, on the best grades and alignments, through the Counties of Tulare and Kings, to the City of Hanford, in Kings County.

28. A highway commencing at a point on the State Highway through Alameda County, at or near Niles, and extending thence southerly, on the best grades and alignments, through Alameda and Santa Cruz Counties to the City of San José; thence southwesterly, on the best grades and alignments, through the Counties of Santa Cruz and Santa Clara, to a point on the highway through Santa Cruz, near Watsonville.

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29. A highway commencing at the City of Sonoro, and extending thence easterly, on the best grades and alignments, through the Counties of Tuolumne and Mono, to a point on the State Highway in the last-named county.

Section 3.—The roads of the second class, or County Thoroughfares, shall be the most important roads in each county, as set apart and so declared by the Boards of Supervisors of the several counties, in discharging which duty they may call upon the Department of Highways, in writing, for such advice and counsel as said Board of Supervisors may desire.

Section 4.—The roads of the third class, or District Roads, shall embrace all existing county roads now recognized and set apart by law, and not enumerated in Sections 2 and 3 of this Act as State Highways or County Thoroughfares, together with such additional roads as may be laid out, in accordance with the laws of the State of California, by the Boards of Supervisors of the several counties.

Section 5.—All Acts or parts of Acts in conflict with the provisions of this Act are hereby repealed.

Section 6.—This Act shall take effect from and after its passage.

The total mileage of roads in the State is about 45 000. This classification would put about 10%, or 4 500 miles, in the first class, and this mileage would ultimately come under the control of the State, and includes all the roads in which the State may be considered to have a direct interest. The remaining mileage would continue under county supervision and control.

ARTICLE III, SECTION 2643, PAR. 7, POLITICAL CODE, TO BE AMENDED AS FOLLOWS:

Cause the road tax collected each year to be apportioned and kept in separate funds by the County Treasurer, as follows:

Fifty per cent. of all moneys so collected shall be apportioned as now provided by law.

The remaining 50% of taxes so collected shall be apportioned to a fund to be known and designated as the "Good Roads Fund." All money so apportioned to the "Good Roads Fund" must be expended by the Boards of Supervisors of the various counties, in the following manner, and for the purposes hereinafter named, and in no other way, and for no other purpose, to wit:

For laying out, grading, sprinkling, graveling or macadamizing the principal highways of the county, and the purchase of all road machinery necessary for the construction of said highways and main-

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tenance of same, and the purchase of water rights, and all necessary property to insure a perfect sprinkling system; for the construction of all substructures necessary to a perfect drainage of a highway or road, all of which substructures shall consist of masonry, concrete, salt-glazed sewer pipe, iron or steel; no lumber or perishable material shall be used, except for bridge flooring, when deemed absolutely necessary.

For better guidance of road construction and the expenditure of moneys in the "Good Roads Fund," the following shall be the order of constructing said highways, and the Boards of Supervisors shall observe this order, so far as possible to do:

First.—The laying out of a highway on the best grades and alignments possible.

Second.—The grading and draining of said highways.

Third.—The construction of permanent substructures.

Fourth.—The establishment of a sprinkling plant for said highways.

Fifth.—The graveling or macadamizing of all such highways.

To perform any work or construct any substructures under this section, the Board of Supervisors to make definite surveys of the proposed work, and to prepare plans, profiles and cross-sections thereof, and to submit the same, with estimate of the amount, or amounts, of work to be done, and the probable cost thereof, and with specifications therefor, duly approved by the Department of Highways of the State of California. The said report shall be prepared in triplicate, one copy to be filed in the Surveyor's office, one in the Department of Highways of said State, and the other to be filed with the Clerk of the Board of Supervisors. The Board, upon receipt of such report, must advertise for bids for the performance of the work specified, as provided in this section. All bidders must be offered opportunity to examine such plans and specifications, and said Board shall award the contract to the lowest responsible bidder, and a copy of the plans and specifications so adopted shall be attached to and become a part of the contract; and the person or corporation to whom the contract is awarded shall be required to execute a bond, to be approved by said Board, for the faithful performance of such contract; *provided*, that after the submission of the bids as herein provided, the Board of Supervisors being advised by the County Surveyor that the work can be done for a sum less than the lowest responsible bid, it shall then be the privilege to reject all bids, and to order the work done, or structure built, by day's work under the supervision and control of said Surveyor; *provided, further*, that the Surveyor in such case shall be held personally responsible, under his official bond, to construct the work and furnish material, at a cost not to exceed the amount of the lowest responsible bid received.

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The furnishing of all material or machinery for the purpose of this Act shall be done by contract and advertisement for bids in the same manner as far as possible as designated herein for road work.

The Boards of Supervisors shall decide all questions pertaining to this Act by a majority vote of all members of the Board.

The County Surveyor and Department of Highways of the State shall have power to inspect any work or advise as to the efficiency or quality of all materials so purchased by the Board of Supervisors for the purposes of this Act, and, upon the written request by the Surveyor and Department of Highways, any failures to comply with the contract or contracts, or any defects in the character of material furnished, shall be remedied by the contractor, and, in default thereof, the Board shall have power to deduct the value of such failure or defect from the contract price agreed to be paid the contractor.

The Board of Supervisors must employ the County Surveyor to superintend the work contemplated by this Act, and, provided said Surveyor is not a salaried officer, must allow him fair compensation for such service.

The Board of Supervisors shall have power to make partial payments upon all contracts let by virtue of this Act, not to exceed 75% of the work done, when the same shall be certified by the County Surveyor as properly performed.

No contract shall be let in conformity to this Act exceeding the amount of money in the "Good Roads Fund" of any county, or the estimated amount to be paid into said fund during the fiscal year in which said contract is entered into.

SECTION 2651, POLITICAL CODE, AMENDED TO READ AS FOLLOWS:

The Board of Supervisors must annually set apart, from the property road tax collected from all sources, 50%, to be set aside as provided in Article III, Section 2643, Par. 7, Political Code; 35% of the remainder of said road tax may be set aside for general county road purposes; from which sum so set apart they may direct such amounts to be paid as may be found necessary for such general county road purposes in which the inhabitants of all the districts within the county are more or less interested, or to assist weak or impoverished districts in keeping their roads in repair, to be applied as the said Board may order or direct; *provided*, that the Boards of Supervisors in the several counties shall have no power to create a debt on any road district in excess of the estimated amount of receipts from said district for the current fiscal year.

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AN ACT TO PROVIDE FOR THE ACCEPTANCE OF HIGHWAYS OF THE FIRST CLASS BY THE STATE, AND THE MAINTENANCE OF THE SAME, AND TO MAKE AN APPROPRIATION THEREFOR.

The People of the State of California, Represented in the Senate and Assembly, do enact as follows:

Section 1.—Whenever five or more consecutive miles of highways of the first class, as defined in an Act entitled "An Act to classify the roads in the State of California, and to define each class," shall have been located and constructed within the limits of any county, in conformity with plans and specifications approved by the Department of Highways, the Board of Supervisors of such county may petition the Department of Highways to accept said portion of road and to maintain the same.

Section 2.—Said Department shall thereupon examine said road and certify to the Governor of the State whether or not such location and construction have been in conformity with the plans and specifications therefor, and if in its opinion said roads should be accepted. Should such opinion be favorable, and meet with the approval of the Governor, such road must then be accepted by the Department in the name of the State, and shall thereafter be maintained by the Department of Highways as provided by the law.

Section 3.—An appropriation of $100 000, or so much thereof as may be necessary, is hereby made for the purposes of carrying out the provisions of this Act during the fiscal years of 1899–1900 and 1900–1901; said fund to be designated as the "State Highway Maintenance Fund."

Section 4.—On or about September 1st, 1900, and biennially thereafter, the Department of Highways must certify to the State Controller the amount necessary to maintain said highways of the first class for the following two fiscal years. The Controller shall include this amount in his estimate of expenditures, and which, when paid into the State Treasury, must be credited to said fund.

Section 5.—The Department of Highways is hereby authorized to advertise for bids, in two newspapers in the county in which the road is situated, for contracts to sprinkle, roll and maintain such State Highways as may be accepted under the provisions of this Act, and for the necessary appliances and machinery for said work. Said Department shall have the power to reject any and all bids, and order the work done by day's labor for an amount less than the lowest responsible bidder, and the Commissioner is responsible on his bond that the work shall be done according to the plans and specifications. Said bids to be filed with the County Clerk of the county in which said highway is situated, and to be opened by the Commissioner of the Department of Highways at the office of said clerk on a day specified in said advertisement for bids.

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Section 6.—The money paid into the State Highway Maintenance Fund is hereby appropriated, without reference to fiscal years, for the exclusive purpose of maintaining State Highways. All claims against the State Highway Maintenance Fund must be made by warrants drawn against said fund by the State Controller, in the name of the person or persons rendering the services or furnishing material provided for by this Act, and must be audited and approved by the Commissioner of the Department of Highways and by the State Board of Examiners. The State Controller is hereby directed and authorized to draw said warrants, when so audited and approved, and the State Treasurer is authorized to pay said warrants, to the extent of moneys available therefor.

Section 7.—This Act shall take effect upon and after its passage.

AN ACT TO PROVIDE FOR THE LOCATION, CONSTRUCTION, AND MAINTENANCE OF HIGHWAYS, OWNED OR TO BE ACQUIRED BY THE STATE OF CALIFORNIA, BY THE LEVY OF A TAX, AND THE CREATION OF A FUND THEREFOR.

The People of the State of California, Represented in Senate and Assembly, do enact as follows:

Section 1.—There is hereby levied annually, for each fiscal year, an *ad valorem* tax of 2½ cents upon each $100 of value of the taxable property of the State, which tax should be collected by the several officers charged with the collection of State Taxes, in the same manner and at the same time as other State taxes are collected, upon all classes or any class of property, which tax is for the location, construction, and maintenance of highways owned or to be acquired by the State of California.

Section 2.—The State Board of Equalization, at the time when it annually determines the rate of State taxes to be collected, must, at the same time, declare the levy of said rate of 2½ cents upon each $100 of value of taxable property, and notify the Auditor and Board of Supervisors of each county thereof.

Section 3.—The moneys collected from said rate, after deducting the proportionate share of expense of collecting from the same, to which other State taxes are subject, must be paid into the State Treasury, and be by the Treasurer converted into a separate fund, hereby created, to be called "The State Highway Fund."

Section 4.—The money paid into said "State Highway Fund" is hereby appropriated, without reference to fiscal years, for the exclusive purposes of locating, constructing, and maintaining the State Highways, and in no case shall any portion thereof be used for the acquisition of title to or rights of way over land or lands, through or

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upon which said State Highways may be located. All claims against said "State Highway Fund" must be audited by the Department of Highways, and approved by the State Board of Examiners, and payments shall be made out of said fund only upon warrants drawn against the said "State Highway Fund" by the State Controller, in the name of the person or persons rendering the services or furnishing material provided for in this Act; and the State Treasurer is hereby directed and authorized to pay said warrants.

Section 5.—No more than 8% of the money derived from said "State Highway Fund" shall be expended in any one county of the State in any one fiscal year.

Section 6.—All Acts or parts of Acts in conflict with the provisions of this Act are hereby repealed.

Section 7.—This Act shall take effect and be in force from and after its passage.

CHAPTER CXVII, STATUTES AND AMENDMENTS, 1897, TO BE AMENDED AS FOLLOWS:

Section 1.—The width of tires for wheels upon wagons or other vehicles to be used upon highways in the State of California shall be, for the following styles of wagons, as follows:

1½ and ⅝-in. steel or iron axle; 2¾ and 3-in. steel or iron thimble skein axle; 2⅛ and 2⅜-in. tubular axle.	Not less than 2½-in. tires.
1¾ and 1⅞-in. steel and iron axle; 3½-in. steel or iron thimble skein axle; 2⅝-in. tubular axle.	Not less than 3-in. tires.

All other vehicles with an axle greater in size or capacity than those above enumerated shall have tires of not less than 4 ins. in width.

Section 3 to be repealed.

Section 4 to be numbered 3.

Section 5 to be numbered 4.

SECTIONS 8 AND 9 OF CHAPTER CCLXXII TO BE AMENDED TO READ AS FOLLOWS:

Section 8.—The Department of Highways shall take possession, in the name of the State, and as rapidly as the funds provided therefor will permit, of all roads which have been or may be declared State Highways, and, in cases where the Legislature may define the general direction and route of a State Highway, the Department shall definitely locate the same upon the best grades and alignments, and as closely following the general lines defined by the Legislature, as the topography of the country will permit, and in consonance with the

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best interests of the State; and whenever the location of an existing county road, or any portion thereof, is such that it may be properly defined as a State Highway, or a portion thereof, then the location of such county road shall be utilized for said State Highway; *provided*, that all highways included within the boundaries of the lands of the State institutions, parks or reservations, now governed by bodies specified by law, are hereby expressly excepted from the jurisdiction of said Department.

Section 9.—The Department of Highways shall have power to locate, construct, and maintain State Highways to the extent of the funds available; and to this end advertise for and let all contracts; purchase, equip, and operate the necessary quarries; and acquire such other property necessary for the construction and maintenance of the roads provided for. Whenever it shall become necessary to construct any part of the System of State Highways, as herein provided for, the cost of which exceeds $500, the Department of Highways must have plans, specifications, and estimates therefor in triplicate, and must advertise for bids for a period of two weeks, in two papers of general circulation, and which shall give the lowest rate for publication, published, one at the point nearest to where the work is to be performed, and one at the State Capitol. Said advertisement shall be in the following form:

Department of Highways, Sacramento, Cal., ..., 18...

Sealed bids will be received by the Department of Highways of the State of California, at its office in the State Capitol, Sacramento, until..... o'clock,.....M., 18.., for....in.....county, California.

Plans and specifications for said work are on file in the office of said Department and in the office of the County Clerk of.....County, at ....., Cal., to which bidders are hereby referred.

[SEAL.]

.....

Highway Commissioner.

These recommendations were not fully considered by the Legislature of 1899. This was a year in which a senator was to be elected, which duty so engrossed the time of the Legislature that but little else was thought of.

Time was found, however, to re-enact the law providing for the construction of an important link of road between the easterly end of the roads leading easterly from the Yosemite Valley and those of Mono County on the other side of the Sierra Nevada.

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The influence of so-called political considerations gave an instructive instance in the drafting of this Act. The roads to be connected are the Tioga Road and those of Mono County, in the basin of Lake Mono. The Tioga Road has been constructed across the Sierras at an elevation of about 9 500 ft., and into the basin of Mono Lake, at the Tioga Mine. From thence three possible routes exist:

First.—Down the cañon of Leevining Creek;

Second.—Northeasterly again over the crest of the Sierras at an elevation of over 10 200 ft., and down the cañon of Mill Creek past a mining settlement at which there are at present concentrated some forty votes;

Third.—To the southeasterly, also over the crest of the Sierras, through Mono Pass, at an elevation of over 10 200 ft., and down Bloody Cañon.

All considerations, economy, shortness of route, lighter grades, freedom from snow, stability of roadbed, suitability of materials, and the best interests of the state, are in favor of the first—the Leevining Creek route.

These facts were fully brought out; yet the legislator who drafted and introduced the bill could not be induced to draw it so as to apply to the best route, but yielded to the importunities of the voters on the Mill Creek route, and drew the bill so that the diversion of the fund to that route could be possible. At this date, it is yet possible that the best of the routes may, through these influences, be abandoned for the worst.*

The attempt to introduce method in road building, however, has not been without result; a much better understanding is had throughout the state, and many practices have been corrected. It must be admitted, however, that political road building so far has had the best of the struggle.

* Since the above was written, the U. S. Commission on Roads in the Yosemite National Park has confirmed the selection of the Leevining Creek route. Pp. 11-14 of Report.

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DISCUSSION.

Mr. Owen. JAMES OWEN, M. Am. Soc. C. E.—This paper is hardly in the line of a professional topic. It recites the history and conditions of road promotion and road legislation in the State of California, and is very interesting, as it gives the experience through which the State of California has gone, which is, to a large extent, similar to that of other States in the Union, except that in California work is not as far advanced as in the Eastern States.

One of the burdens of Mr. Manson's subject is the cry that many engineers make, viz., the interjection of the political element into road work.

Attention should be called particularly to the clause on page 347, because the speaker has never before seen it in the shape of a legal enactment. If it were in active practice in this section of the United States it would devolve upon the engineer to determine whether a bid was too high, and if it should turn out that the work cost more than he estimated, he would have to pay the difference out of his own pocket. It is a question whether the engineering profession at large would like to have such a practice incorporated in their ethics.

There is one point about the California work that does not seem to be stated by Mr. Manson, and that is, that the State Penitentiary, in the early incipiency of the road work of that state, put up a large stone-cracking plant, and at that time a very advantageous rate was made with the Southern Pacific Railroad to haul the road material to different sections of the state. The state engineer at that time told the speaker that they were hauling road material at a very moderate cost for a distance of over 600 miles. The speaker does not know whether or not that practice is kept up now, but it was a very important procedure. The railroads there felt that highway construction was an adjunct to their business, and offered these inducements to facilitate the further development of road work. The railroads in the East, so far, have not taken that view of the road work, but have always demanded full rates for their mileage, in common with other fields of enterprise.

There is another interesting phase of Californian legislation which differs materially from the practice in the East, and it has a great deal of merit: The law provides that a sum of one-half the road tax shall be reserved for what is called a "good-roads fund," out of which the cost of the road construction is paid. After the road is built it is turned over to the state for maintenance, and the state, under the statute, must provide and appropriate $100 000, or so much thereof as may be necessary, for the purpose of carrying out the provisions of that act. The practice in the East is entirely different. Where the states have interjected their authority and funds

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for the construction of a road, the provision afterward is that the local authorities shall pay for the maintenance—that is, as far as the speaker knows—he does not remember exactly the practice now in Massachusetts—whether or not the State Road Commission there provides a fund for the maintenance—but in New York, New Jersey, and the States with which he is cognizant, the local authorities have complete charge of the expenditure for maintenance. Mr. Owen.

The speaker is inclined to think that that Californian provision is an improvement. The more concentrated the authority of maintenance the better will be the result. The difficulty always found in road construction or road maintenance is, that while people are very anxious to have good roads, and appreciate them, the local authorities are not anxious to appropriate the necessary moneys to keep them in good repair.

GEORGE W. TILLSON, M. Am. Soc. C. E.—In Toronto, Canada, the practice in receiving bids is somewhat similar to that mentioned in the last clause of the California law. When bids are asked for in Toronto, or rather when they are received, the practice is for the city engineer himself to put in a bid on the work. If that bid is lower than any of the others received, the work is awarded to him and he completes it by day's labor. The object, of course, is not to give the city engineer the opportunity to pay for anything out of his own pocket, but is to prevent the bidders from making a combination and getting more than a reasonable price for the work, because it ought not to be difficult for the contractors to do work at a fair profit, but for less than city officials, in this country, at least, could do it by day's labor. Mr. Tillson.

In looking over the report of the city engineer, where he has reported upon work for which he put in his proposal and was awarded the work, it is found that, as a general rule, he has often done the work for less than the amount of his bid, but in some few cases the cost of the work has exceeded his bid.

In regard to the Massachusetts law, the President of the Commission told the speaker, a short time ago, that at the present time they have charge of the maintenance of the roads, although at first they did not; and that the cost last year averaged $95 per mile for maintenance, over all the roads, although some of the roads, on which the work was done by contract, were kept in good repair for $70 per mile.

MARSDEN MANSON, M. Am. Soc. C. E. (by letter).—In the application of the principles of engineering to the varied problems before that profession, the physical, economic, legal and political phases have to be broadly construed and balanced. When the work or problem has long been treated without any professional consideration whatever, the history of this treatment and its results must enter into the engineering study. The writer does not agree, therefore, with Mr. Owen, that a brief consideration of road history and legislation are Mr. Manson.

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Mr. Manson. not professional topics; they are all elements in the great problem of rectification which, sooner or later, has to be undertaken.

The principles of engineering must not only be applied to re-location, drainage, grading and installing a true highway, but, for considerable periods, must be also applied to the maintenance of the next to impassable surfaces called roads.

The writer overlooked the excellent work done by the rock-crushing plant at the State Penitentiary at Folsom. Mr. Owen, however, was misinformed as to the distances over which its product was distributed. Small quantities were sold to adjacent counties at cost, and transported at very cheap rates to perhaps one-fourth the distance mentioned by Mr. Owen, or about 150 miles as a maximum. The greater part was taken by the cities of Stockton and Sacramento. The plant was destroyed by fire, supposed to have been incendiary, but has been re-established on a diminished scale.

The provision requiring one-half the road funds to be set aside as a Good Roads Fund was one of the recommendations of the Highway Commission; it did not become a law, as Mr. Owen seems to infer. A careful reading of the paper will correct this misapprehension; nor was the recommendation carried out that $100 000 be appropriated as a Maintenance Fund.

The provision in the Canadian law, mentioned by Mr. Tillson, permitting the engineer to bid in the work and perform the same by day's labor, is an excellent one, as it provides an adequate check on collusion among bidders.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 927.

LINE AND SURFACE FOR RAILWAY CURVES.*

By Charles C. Wentworth, M. Am. Soc. C. E.

With Discussion by Messrs. James K. Geddes, Charles A. Morse and Charles C. Wentworth.

It is safe to say that there are in existence thousands of miles of railway track the original center line of which may be described, for the purposes of this paper, as a series of tangents connected by circular curves. Right of way was secured and the earthwork completed, using this center line as the base of measurement; and, finally, track was laid to the same center line and the whole turned over to the operating department.

A subsequent retracing of the original center line would show that it had been found necessary, or at least advisable, to deviate from it materially in order to secure the smooth riding of trains. Short tangents, connecting curves turning in opposite directions, have been shifted at each end, away from the centers of these curves, thus increasing the total amount of curvature; or the points of curve and tangent have been moved closer together, thus decreasing the length of the straight track between them. In the latter case it is not unusual that the original curve has been materially sharpened at the end of the flat curve introduced; and cases have been observed in which

* Presented at the meeting of April 2d, 1902.

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the tangent was deflected at a point a short distance from the original P. C., in order to secure a lighter approach to the original curve.

This paper is an attempt to indicate what was wrong with the original center line, and to propose simpler means for correcting the trouble than those now in use.

The conditions to be met are:

First.—The method must be of the simplest kind that will be theoretically exact, and, as a consequence, practically so. If the most gifted calculator had to perform in the field, and with half-frozen fingers, the operations called for by some methods he would appreciate the importance of this first condition.

Second.—The position of the original tangents must not be altered. It is impracticable to shift long tangents, and the shifting of short tangents involves such a change in the angles of intersection that the problem becomes indeterminate.

Third.—The length of the center line must not be altered by the revision. Grades, distances and right-of-way matters require this. A multiplicity of long and short stations is apt to be a never-ending source of future uncertainty.

Fourth.—The resulting center line, and the changes in the elevation of the outer rail involved by the revision, must be such as can be followed exactly in practice, without such practical alterations as are sometimes found necessary for successful operation.

The principles involved may be stated as follows: In order to impart a given rate of motion to a heavy body, a force must be used. The smaller the force, the less will be the jar caused by it while acting. The smallest force that will impart the given motion is the one that acts uniformly during the entire given time. The total amount of motion imparted by such a uniform force is proportional to the square of the time, and the rate of motion is proportional to the time.

The motions imparted to a car leaving a tangent and entering a curve are two in a horizontal and one in a vertical plane. To these motions the principles given hereinbefore are to be applied. Each motion being taken care of separately, the result will be the best that can be gotten out of the situation.

The first motion in a horizontal plane is one about the center of the curve. Inasmuch as on a circular curve the departure from a tangent

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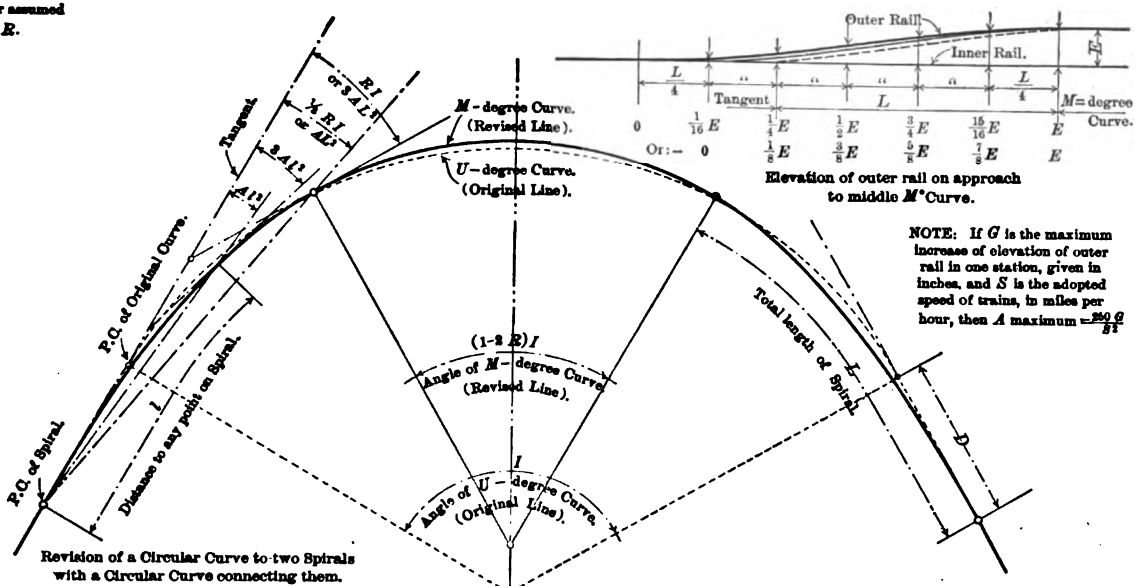
Values of B for assumed values of R.

R	B
$\frac{1}{12}$	1.0128
$\frac{1}{10}$	1.0133
$\frac{1}{9}$	1.0225
$\frac{1}{8}$	1.0333
$\frac{1}{6}$	1.0486
$\frac{2}{10}$	1.0705
$\frac{1}{4}$	1.1000
$\frac{2}{10}$	1.1384
$\frac{1}{3}$	1.1856
$\frac{4}{10}$	1.2408
$\frac{1}{2}$	1.3000



Revision of a Circular Curve to two Spirals with a Circular Curve connecting them.

Position of tangents and length of center line unchanged.

Given:- U and I

Assumed:- $\frac{\text{Tangential angle of each spiral.}}{I} = R$

then $\left.\begin{array}{l} M = BU \\ D = (R + \frac{1}{2})\frac{I}{M} - \frac{I}{2U} \\ A = \frac{M^2}{12RI} \\ L = \frac{2RI}{M} \end{array}\right\}$ in which $\left\{\begin{array}{l} A = \text{deflection for the first 100 ft. on spiral.} \\ B = 1 + 1.896R^2 - 0.672R^3 \end{array}\right.$

All distances in 100-ft. stations, and all angles in degrees.

Elevation of outer rail on approach to middle M° Curve.

NOTE: If G is the maximum increase of elevation of outer rail in one station, given in inches, and S is the adopted speed of trains, in miles per hour, then A maximum $= \frac{250\,G}{S^2}$

Properties of Spiral:

Distance along spiral.....	1.00	2.00	3.00 etc.	l	L
Deflection from tangent...	A	$4A$	$9A$	Al^2	$\frac{1}{2}RI$
Tangential angle.....	$3A$	$12A$	$27A$	$3Al^2$	RI
Degree of curvature	$6A$	$12A$	$18A$	$6Al$	M

FIG. 1.

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at a given point is initially proportional to the square of the distance run, this motion requires no treatment, except that, in order to make the deviating force as small as possible, the amount of deviation should also be as small as possible, or the radius of curvature as great as possible, a fact which is well recognized.

The second motion in a horizontal plane is one of each car about its own center, as if it were mounted on a turn-table. In order that the force producing this motion be as small as possible, as stated hereinbefore, the amount of angular deviation must be proportional to the square of the distance run; the speed along the track being uniform. This may be accomplished by using such an approach curve that its tangential angle at any point is proportional to the square of the distance of such point from the P. C., measured on the arc.

The motion in a vertical plane is that caused by the deviation of the car from the vertical, due to the elevation of the outer rail. If it be assumed that the angular departure from the vertical is proportional to this elevation, then this elevation must vary as the square of the distance run until a certain rate of motion is attained. The line of increase in elevation may then become straight, continuing so until it is necessary to stop the motion, when the motion is to be eased off in the same manner that it was created.

Revisions of a center line, as herein indicated, can be made much more advantageously prior to the construction of a railway than subsequently, as then they can be made on a scale commensurate with the dignity of the undertaking. There is no better way to project locations of railway, in a difficult country, than with circular curves on contour maps. In revising the center line of existing track, circular curves are the subjects of revision. A circular curve has then been made the basis of revision on the lines suggested, and the results obtained, all of which are in strict accordance with the conditions and principles before mentioned, are shown in Figs. 1 and 2.

These figures are intended to be clear as to their presentation of the subject, though necessarily any new method requires some additional explanation. The following examples will make the matter quite clear:

Fig. 1 treats of such revisions of a circular curve as require a middle circular curve to connect the spirals.

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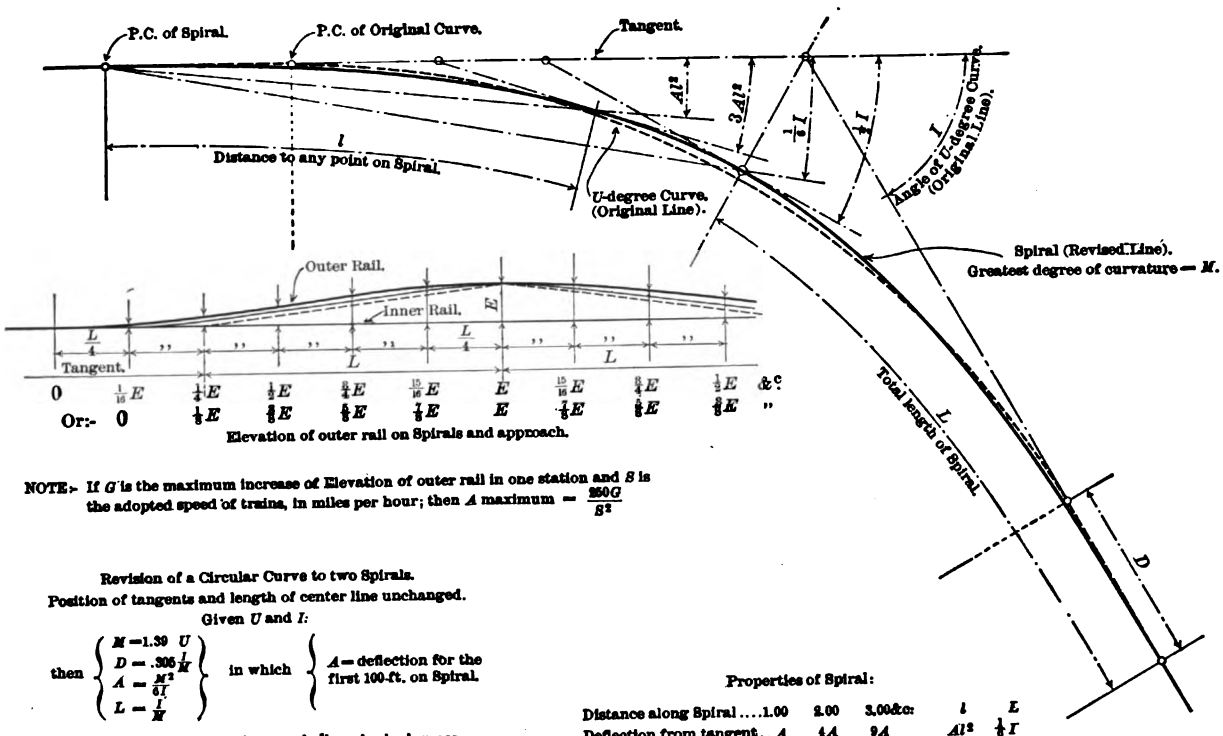

Elevation of outer rail on Spirals and approach.

NOTE:- If G is the maximum increase of Elevation of outer rail in one station and S is the adopted speed of trains, in miles per hour; then A maximum $= \frac{250G}{S^2}$

Revision of a Circular Curve to two Spirals.
Position of tangents and length of center line unchanged.
Given U and I:

then $\left\{ \begin{array}{l} M = 1.39\ U \\ D = .305\frac{I}{M} \\ A = \frac{M^2}{6I} \\ L = \frac{I}{M} \end{array} \right\}$ in which $\left\{ \begin{array}{l} A = \text{deflection for the} \\ \text{first 100-ft. on Spiral.} \end{array} \right.$

All distances in 100-ft. stations, and all angles in degrees.

Properties of Spiral:

Distance along Spiral	1.00	2.00	3.00&c:	l	L
Deflection from tangent...	A	$4A$	$9A$	Al^2	$\frac{1}{6}I$
Tangential angle	$2A$	$12A$	$27A$	$3Al^2$	$\frac{1}{2}I$
Degree of curvature.....	$6A$	$12A$	$18A$	$6Al$	M

FIG. 2.

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Fig. 2 considers curves made up of spirals only, as would be the case in short curves generally, or in comparatively long curves when liberally treated, and when the increase of curvature (39%) does not prohibit.

There may be, for instance, a 6° curve for twenty stations, the total angle of intersection being 120 degrees. If the tangential angle of each spiral be assumed as being 30°, then $R = \frac{1}{4}$, and $B = 1.106$; so that, for the degree of curvature for the middle curve, $M = 6.648°$, or 6° 39′.

For the distance from the P. C. of the original curve to the P. C. of the spiral, measured on the tangent, $D = 3.538$, or 353.8 ft.

For the angle of deflection from the original tangent, to be used to set a stake 100 ft. from the P. C. of the spiral, $A = 0.1228$, or 0° 7.37′.

For the length of the spiral, $L = 9.025$, or 902.5 ft., all being figured from the formulas given in Fig. 1.

As this spiral extends through nine stations, the deflections, for each station, successively, are 0° 7′, 0° 29′, 1° 6′, 1° 58′, 3° 4′, 4° 25′, 6° 1′, 7° 52′ and 9° 55′. Then the deflection for 902.5 ft., or the total length of the spiral, is one-third of the assumed tangential angle, or 10°, and the tangential angle 30°, as assumed. Spirals can be run in thus from both tangents, and then connected by a 6° 39′ curve between them for the remaining 60° of total angle.

It may be noted that the length of the revised line is 2707.5 ft., as compared with 2707.6 ft. by the original line, measured in both cases between the P. C. and P. T. of spirals. Also, that both lines, as projected on the long chord connecting the P. C. and P. T. of spirals, agree as to length, 2008.6 ft. The second and third conditions, therefore, are complied with.

If the spirals in this case be thought unnecessarily long, the tangential angle of each spiral could be assumed as being, say, 15°, in which case $R = \frac{1}{2}$. Then, from Fig. 1,

$$M = 6.170°, \text{ or } 6° 10';$$
$$D = 2.156, \text{ or } 215.6 \text{ ft.};$$
$$A = 0.2115, \text{ or } 0° 12.7';$$
$$L = 4.862, \text{ or } 486.2 \text{ ft.}$$

It may be desired to revise a circular curve in such manner that the revised line will be all spiral, as shown in Fig. 2. In a 10° curve for 600 ft., or 60°, we have, by substitution.

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$$M = 13.9°, \text{ or } 13° 54';$$
$$D = 1.316, \text{ or } 131.6 \text{ ft.};$$
$$A = 0.5367, \text{ or } 0° 32.2';$$
$$L = 4.316, \text{ or } 413.6 \text{ ft.}$$

The theoretical elevation, in inches, E, for a given degree of curvature, D, and speed, in miles per hour, S, may be found by the formula, $E = \frac{S^2 D}{1500}$. It is not the province of this paper to attempt to prescribe a certain elevation for a given degree of curvature and speed, because there is room for a considerable difference of opinion on this subject.

This is due to the fact that it is the manner of attaining an assumed elevation, and the uniformity with which it is maintained when reached, that is of prime importance, rather than the absolute elevation itself. This will be evident from the consideration of a piece of straight track, perfect in all respects except that one rail is 1 in. lower than the other for a space of, say, a mile. This will be the best riding piece of track on its division—better perhaps than if the rails were on the same level—because the wheel flanges will run close to the low rail rather than deviate from rail to rail, while the difference of elevation will be almost, if not quite, imperceptible to anyone in an ordinary car. If this be granted, and it will be found correct, 1 in. more or less than the theoretical elevation cuts no figure, as far as smooth riding is concerned, after proper approach curves have been provided.

It will be seen from Figs. 1 and 2 that elevation is given the outer rail at the P. C. of the spiral, notwithstanding the fact that the radius of curvature there is infinite. This is done because a car is not a rigid body with respect to its trucks, and, in order to impart a given motion to the body of the car, such motion must be imparted to the trucks in advance of the time when it will be required by the car itself. Further, it will be seen that it is impossible to provide proper vertical curves at the ends of the grade of elevation unless either an elevation of the outer rail be given at the P. C., or the rate of grade increased. The vertical curves used in the easements to the elevation grades in the illustrations are common parabolas.

If a point be taken on a given tangent, and a polygonal segment, with 100-ft. sides, be staked out therefrom, such that the angle between the first side and the tangent is A degrees, the angle between the tan-

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gent and the end of the second side is 4 A degrees, or, to the end of n sides, $n^2 A$ degrees, and pass a curve through the apices of this polygon, the result will be the spiral of this paper. This curve is not a cubic parabola, but departs from its tangent at a more rapid rate. Consequently, the properties ascribed to it in Figs. 1 and 2 can be so ascribed with much more accuracy than would be the case with the cubic parabola. It fulfills accurately the assumed conditions, and is capable of being harnessed in such way that the resulting formulas are much simpler, being free from even the ordinary trigonometrical functions, as may be seen from the illustrations.

The writer will not herein go over the reductions necessary for the production of the given formulas, but will leave them, in the hope that they may be found useful in a field where little has been done, though much is desired.

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DISCUSSION.

Mr. Geddes. JAMES K. GEDDES, M. Am. Soc. C. E. (by letter).—The transition spiral described by Mr. Wentworth seems to be well adapted, indeed, to the revision of lines where the track has already been laid, in that, as a rule, it does a minimum amount of violence to the original location. It would seem to be much less complicated than such spirals as the Searles, and fully as easy to stake out when once the necessary computations are made.

However, like nearly all transition curves so far presented to the professional public, it seems to be objectionable on account of the difficulty of calculating and laying out the revised line, and keeping the track to proper alignment after the change has been made. Few railways in this country, as yet, can keep a force of engineers in the field all the time giving centers to the track men, and such curves, even when monumented, would be difficult or impossible to keep in proper alignment by the average track force. If the transition curve could be made comparatively short, and simple tables and rules worked out for the cases likely to occur in practice, so that, when once located, the section men could keep the curve in proper alignment, much would be done to popularize this desirable curve. As it is, the average railway man—whether he be an engineer or not—with his strenuous life, is very prone to fight shy of transition curves, with a lot of formulas that are simple enough, perhaps, had he the time to consider them carefully as to their adaptability to his particular needs, and had he track men who could handle the curves as they do other routine track matters.

Mr. Morse. CHARLES A. MORSE, M. Am. Soc. C. E. (by letter).—The author shows by his diagrams that he carries the low rail at the grade line and puts all the elevation in the outer rail. The Committee on "Track," in the Third Annual Convention of the Maintenance Association, made a similar recommendation, and it was accepted by the Convention without a word of discussion, which would seem to indicate that it was the practice on most railroads, or that the matter had been given little thought.

The practice originated with track men, when engineers on maintenance were not employed to any extent, and, under the circumstances, was the best they could do, as they had to have one rail to grade, to surface the track; but even then it was recognized that it increased the curve resistance, and this was prevented, by some track men, by running the outside or upper rail to grade and lowering the inside rail to get the elevation.

This could not be done without a good deal of expense, except in cases where the track was being raised several inches, and the result was the general use of the inside rail as a grade rail.

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Mr. Morse. The cause of this practice, however, has disappeared on the larger roads at the present time, as engineers are considered as a necessary part of the maintenance force, and are called upon to re-center curves whenever they get to riding poorly, and also to set stakes for the proper elevation of the "outer rail." Everyone who has had occasion to plot a profile of a piece of track for the purpose of laying ballast grades has noticed the "hump" in the surface of the plotted center-line elevations, caused by all the elevation being given to the outer rail, and could not fail to see the effect on the train resistance at that point.

The grade is compensated on curves to offset the curve resistance, but, when all the elevation is given to the outer rail, the first 200 or 300 ft. of the curve has really a heavier grade than the maximum on the tangent next to it.

The result of this practice is the too common idea that the train resistance is increased for slow trains, when the track is elevated for fast passenger service, but the fault is in the way in which the elevation is put up, and not in the fact that there is elevation given.

Elevation on curves should be divided. The outside rail should be elevated one-half and the inside rail depressed one-half of the full amount of elevation required for the speed of the fast train, and in this way the center of the track is kept at the true grade, and the loaded car does not have to be raised, but follows the compensated grade line, as laid down on the profile.

While stakes are being set, it is as easy to set them this way as the other, and they would not require re-setting any oftener than the easement curves would require re-centering, and this could be done at the same time.

While, perhaps, it is not advisable to lower track to get the depression for the inside rail, yet most roads are constantly raising track, and the desired result could gradually be obtained by holding the inside rail down and not lifting it at all, and in this way, in two or three surfacings of the curve, the elevation would be divided so that half would be above and half below the grade line. On roads where ballasting is being done, there is no trouble in staking it at once.

Why this old track man's expedient for elevating track should have survived and be recommended in this day of improvement, and especially when tonnage is the great question of the day, can be explained only by the fact that it is one of the small things which have not been looked after, but when it is given as good practice, by maintenance associations and writers on track work, it is time to call a halt and look into the matter.

To illustrate the effect, on a tonnage train, of the full elevation being given to the outer rail, take the case of a 2° 30′ curve on a maxi-

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mum grade of 0.80 ft., which is put up for a passenger train speed of 60 miles per hour. The outer rail would be elevated 6 ins., and the center of the track would be raised half of this, or 0.25 ft. Now, if the easement is 200 ft. long, the rate of the grade on the first 100 ft. of easement would be 0.885 ft., and if the grade was compensated 0.05 ft. per degree, the rate on the second 100 ft. would be 0.845, whereas the profile grade on the first 100 ft. is 0.76 and on the second 100 ft. 0.72; in other words, the maximum grade is exceeded at the very point where compensation is required. Mr. Morse.

A train coming upon a curve put up in this way, when its speed is reduced to 10 or even 6 miles per hour, and it is pulling every ton it is rated for, feels this increase of grade, and is slowed up and often stalled before getting far on the curve. The lighter the maximum grade, the greater is the resistance to tonnage trains when all the elevation is obtained by raising the outer rail.

The increase of the grade on the easement to a curve is the more objectionable from the fact that it is on this part of the curve that the greatest resistance is felt from the friction of the body of the car on its side bearings. As soon as the elevation begins, the center of gravity of the load is moved toward the inside rail, the load comes down on the side bearings on that side, and as the radius of the curve on the easement is constantly changing, so is the angle between the truck and the body of the car, and the pressure on the side bearings tends to prevent this movement and retard the speed of the train. After the car gets on the regular curve, the angle of the truck remains constant, the movement on the side bearings is small and its effect is less.

That this advantage to tonnage trains is recognized and taken into account is shown by the fact that one of the great systems of the country, which is spending large amounts in grade reduction to help its train-haul, and which is building several hundred miles of new road, is giving the surface of its banks on curves an elevation of 2% of the width of roadbed per degree of curve, raising the outside one-half and depressing the inside one-half, thus making the center of the roadbed throughout on profile grade.

CHARLES C. WENTWORTH, M. Am. Soc. C. E. (by letter).—Mr. Geddes mentions favorably a feature of the proposed method of relieving circular curves: That, generally, it does a minimum of violence to the original location. This is due to the length of the center line being unchanged. Mr. Wentworth.

There is another feature due to the same cause: That such revisions, when applied to a preliminary location, will result in a line that will almost invariably fit the ground better than the original simple circular curves. This is true whether the original line be one seeking low ground over a point or high ground in a ravine: The departure

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Mr. Wentworth.

from the original circular curve is in the right direction at its P. C., middle, and P. T.

Still, as Mr. Geddes seems to doubt the desirability of such a treatment of circular curves, a few words on its practical application, and the advantage to be derived therefrom, may not be out of place.

On one hand there is what may be called current practice. Given, a circular curve, with its tangent, staked out on the ground (as every curve must be at some period of its existence), this given center line is not followed when the track is being put in final condition for the passage of the trains, but the points of curve and tangent are shifted from one to ten rail-lengths in order to introduce that necessary easement which may be called the "track man's spiral." Finally, elevation is given the outer rail, by general rule, fully at the point of curve. Which point of curve?—the original one, the final one, as fixed by the trackman, or some intermediate point?

It will be seen, then, that under this procedure the final outcome in a given case cannot be fully foreknown. As a practical example of this, see the discussion by Mr. Boggs* on Mr. Lee's paper on "Transition Curves." Nevertheless, when tempered by judgment and experience, current rules applied to such work do secure a fairly good result; otherwise, so-called circular curves would have been abandoned long ago.

A partial analysis of this result may be made as follows: If a car were to pass quickly from a tangent to a circular curve accurately laid to the center line, but without elevation being given to its outer rail at its P. C., the outward lurch, especially at the forward end of the car, would be great. A contrary lurch, of any desired magnitude, can be secured by elevating one rail on straight track. A combination of these two opposing lurches may produce a fairly steady car, much in the same way that two properly superimposed sound waves produce silence. Just how much elevation of the outer rail is required, in any particular case, at the end of the tangent or the beginning of the curve, depends on the essential features of the "track man's spiral"; but such elevation is generally considerably greater than would be necessary on the main curve, if a well-considered transition spiral had been introduced.

These considerations may be exemplified further by the result attained when the end of a circular curve occurs on a bridge. Here limitations are imposed as to the lateral departure from the projected center line; and the elevation of the outer rail is given, from rules, by the bridge carpenter. Final adjustment by the track men being difficult, or impossible, the result is seldom as good as if the rails were on ties and ballast.

On the other hand, there may be a center line staked out consisting of circular curves connected with their tangents by well-proportioned

* *Transactions*, Am. Soc. C. E., Vol. xlvi., p. 396.

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spirals. On laying track there will be no desire on the part of the track men to deviate therefrom; as such a center line is what they have striven for, and obtained partially, since railways began. Monuments, then, are even less necessary than before. A certain displacement is avoided, and possible displacements are no more apt to occur, or more injurious if occurring. Mr. Wentworth.

If the elevation of the outer rail be thought difficult of proper attainment on spirals, it will be granted that on the middle circular curve, at least, the problem is unchanged. If an elevation of 1 in. per degree be established, then, on the main curve, the middle ordinate to a chord 61.8 ft. long will equal the required elevation. If 2 ins. per degree, the length of the chord is 87.4 ft. For e inches elevation per degree, the length of the chord, in feet, is $61.8\sqrt{e}$.

The elevation per degree of curve being assumed, and the corresponding length of chord known, this same chord will determine the elevation of the outer rail at any point on the spiral. The spiral is readily recognizable by its uniformly increasing ordinate, and, in the space occupied by it, an extra elevation (say $\frac{1}{2}$ in.) is to be provided, for reasons given in the paper; but the elevation assumed for the main curve is nowhere to be exceeded. Even by such a practical treatment of spirals as this, better and more certain results will be attained than by what has been herein (perhaps inadvisedly) called current practice.

In bridge work, or other work wherein the exact position of the center line and elevation of the outer rail are required to be accurately foreknown, the writer's method allows of such determination precisely.

The writer does not agree with Mr. Geddes that what is needed is a short spiral fixed by tables. There is no more reason (apart from the labor) for a uniform spiral than for a uniform degree of curvature for all main curves. Acting on this, it has been the endeavor of the writer to simplify means for a general solution of the problem, whereby the engineer's spiral may be made as long as necessary for the complete development of the "track man's spiral," aforesaid.

As Mr. Morse says, the maintenance of the low rail at grade, and the elevation of the outer rail, affects the motion of trains. Whether such is sufficiently prejudicial to cause the depression of the inner rail one-half—ordinarily a difficult and expensive performance—may be judged, in any given case, on such lines as the following:

A train weighing 1 000 tons, and moving at a speed of 10 miles per hour, has necessarily stored in it 6 680 000 foot-pounds of energy. If his 2° 30′ curve be taken as an example, with its elevation of 6 ins. in the outer rail only, the energy required to lift the center of gravity of the entire train 3 ins. is 500 000 foot-pounds. The remaining energy assuming that the engine could maintain the speed of 10 miles per hour, save for the resistance due to the manner of elevating the outer

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Mr. Wentworth.

rail only, is 6 180 000 foot-pounds, which corresponds to a speed of 9.6 miles per hour.

This reduction of speed is gradual, and is finally accomplished at the time when the caboose at the rear of the train reaches the summit of the elevating grade. In other words, the final reduction, of only $\frac{1}{10}$ mile per hour, is reached just when the obstruction is passed.

The same amount of energy (500 000 foot-pounds) is imparted to the train again on reaching the descending grade at the P. T.; so that the net result of this pair of velocity grades, as they properly are, is zero; and an argument against their use becomes rather one against the somewhat prevalent practice of overtaxing that willing horse, the locomotive engine; considering that the lowering of the inner rail be attended with any difficulty or expense whatever.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 928.

THE STIFFENING SYSTEM OF LONG-SPAN SUSPENSION BRIDGES FOR RAILWAY TRAINS.*

By JOSEPH MAYER, M. Am. Soc. C. E.

WITH DISCUSSION BY MESSRS. L. S. MOISSEIFF, H. A. LA CHICOTTE, W. HILDENBRAND, L. L. BUCK, EDWIN DURYEA, JR., AND JOSEPH MAYER.

The improvements in the quality of the materials used for the construction of bridges have gradually increased the largest practicable distance between the piers, or the length of span. This is from 200 to 300 ft. for stone, concrete and timber bridges, and a maximum of 2 000 ft. for bridges of the best structural steel now obtainable.

The progress in the manufacture of steel wire has made it practicable to construct bridges of a span of over 3 000 ft. Long-span bridges generally cross navigable waters, where the difficulty of erection limits the choice to cantilever or suspension bridges. Different locations offer unequal advantages for the two types. In some locations natural rock can be used for the anchorages of a suspension bridge; in others, these anchorages have to be built of concrete or masonry on deep and expensive foundations. It is, therefore, impossible to fix accurately the limits of span for the economical use of the two types without investigating the proposed site.

* Presented at the meeting of April 16th, 1902.

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Another factor of prime importance, extending or limiting the field for the economical use of suspension bridges, is the nature of the moving load and the permissible maximum grade on the structure. If the moving load consists of street cars, wagons, carriages and people on roadways and sidewalks, the bridge will generally be nearly uniformly loaded over its whole length, and a grade of 3% is unobjectionable. If the bridge is for railroad trains the moving load will often cover only a part of its length, the other part being entirely unloaded, and a grade of more than 1% may be very objectionable.

The unequal loading of different spans, or of different parts of the same span, of a suspension bridge, is the most important cause of its deformation. The permissible maximum grade is the main factor that fixes the allowable deformation.

To diminish the deformation of a suspension bridge under unequal loading, a stiffening system is required. This will be heavy when there is very unequal loading over the length of the bridge and when the permissible deformation is small. It will be light when the load is nearly uniform over the length of the bridge and when the permissible deformation is great. The former is the case for railroad bridges; the latter, for highway bridges. This is the main reason why highway suspension bridges are economical for nearly all spans when the site is favorable for the anchorages, while railway suspension bridges are economical only for very long spans.

When the span exceeds 800 ft. the conditions must be extremely unfavorable for the anchorages to make a highway suspension bridge uneconomical. When the span exceeds 1 500 ft. a railway suspension bridge becomes economical with natural rock anchorages. When the span exceeds 2 000 ft. the suspension bridge is the only practicable type.

The essential feature characterizing and distinguishing a long-span suspension bridge consists in one or more carrying cables, of steel wires or wire ropes, attached to anchorages at the ends and passing over supporting towers. These cables take the place of the tension chords of a truss bridge, while the compression chords are replaced by the anchorages. The web system of a truss bridge is replaced, either by suspended stiffening trusses, or by bracing between cables placed above each other in vertical or nearly vertical planes. The cables in the latter case are the chords of the stiffening trusses.

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The cables between the towers and the anchorages may be used to support the shore spans, as in the existing New York and Brooklyn Bridge, or the shore spans may be either partly or wholly supported by piers. The cables, in the last case, pass in nearly straight lines from the towers to the anchorages. There may be moving or fixed saddles on the top of the towers to receive the cables. In the latter case the towers oscillate with their expansion and contraction. The cables may be in vertical planes or in slightly inclined planes; in the latter case they are said to be cradled.

The stiffening trusses may be continuous at the towers and in the center of the main span, or they may have two or three hinges; they may have a depth equal to one-twentieth of the length of the main span, or their depth may be only one-one hundredth, or less, of this length. There may be two or more stiffening trusses.

Other methods of stiffening long-span suspension bridges are possible, but will not be considered, as their inferiority is now generally conceded.

Suspension bridges for railways differ from those for highways by the greater amount of the forces tending to produce deformation, and by the smaller deformation that is permissible. Their stiffening system, therefore, is of much more importance than that of highway suspension bridges. It is, at the same time, the feature about which there is the greatest difference of opinion among engineers, showing itself in fundamental differences in the designs proposed. The writer, therefore, has selected the stiffening system for discussion. Only two types of stiffening systems will here be considered: Suspended stiffening trusses, and braced cables. For both of them the questions arise: Whether to use hinges or not; and the proper depth between the chords of the stiffening trusses.

Suspension bridges, especially those with braced cables, have many features in common with arches. In both suspension bridges and arches the end reactions have a horizontal component; in truss bridges, they are vertical. While suspension bridges tend to pull their anchorages toward each other, arches tend to push their abutments apart. The calculations of the stresses of arches and of the main span of suspension bridges with braced cables are nearly alike. Inferences, therefore, have been drawn from the general dimensions of arches that have proved satisfactory to the best general dimensions of suspension bridges.

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Two very important differences between long-span suspension bridges and arches, however, have a direct bearing on the determination of the best general dimensions of the former. The ends of the main span of a suspension bridge are supported on high towers and connected with the anchorages in their rear by elastic cables. These ends, therefore, are far from fixed in position. The ends of arches are practically fixed. The cables of long-span suspension bridges, corresponding to the chords of a steel-arch bridge, are made of wire having an elastic strength three to four times as large as that of the structural steel of the chords of the arch. The permissible unit stresses in the cables and the consequent deformations, therefore, are very much larger than the unit stresses in the chords of an arch bridge and its deformations.

These two differences between arch and suspension bridges, both having the effect of largely increasing the deformations of a suspension in comparison with that of an arch bridge of similar general dimensions, are evidently important in determining those general dimensions which will keep the deformations within given narrow limits. No reliable inferences, therefore, can be drawn from the proper general dimensions of arches to those of suspension bridges.

The deformations of ordinary truss and cantilever bridges, produced by loads and changes of temperature, are of little importance in determining the general dimensions; they become important in arches with less than three hinges, and they are the governing factor in suspension bridges.

The advantages of the various types and general dimensions of suspension bridges can only be shown on concrete examples. There exist no suspension bridges of the kind here considered. A great amount of thought, however, has been expended to develop a satisfactory design for a suspension bridge for railroad trains across the Hudson River, in New York City.

These designs furnish the concrete material on which the arguments for the various types can be tested. Such a bridge should enable the long-distance passenger trains of the railroads having terminals across the Hudson to come to a central station in the hotel and residence district of New York City. It should enable the New York Rapid Transit trains and the surface cars to proceed to the suburbs in New Jersey so as to give direct communication, without change, between

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all parts of New York City and the New Jersey suburbs. It might also be used to some extent by freight trains, especially if a proper freight approach can be secured in New York City. It is evident that many tracks will be required to perform this service.

There will be needed: Two tracks for freight trains, if freight business is contemplated; six tracks for accommodating the suburban traffic, mostly with lower New York (this traffic would use the elevated and underground roads); two tracks for surface cars connecting the near regions east and west of the bridge; two tracks for long-distance passenger trains going to a central station.

The bridge, therefore, should have ten passenger tracks, and perhaps two freight tracks.

For the purpose of comparing the merits of various types for a bridge across the Hudson, the writer, therefore, will use a bridge having twelve tracks, of which two are for freight trains, two for long-distance passenger trains, six for trains of the elevated and underground roads of New York City and two for surface cars.

Such a bridge has sufficient capacity for the business it can secure in the near future. The distant future may bring other competing bridges or tunnels, therefore it need not be considered.

The moving load would consist of two freight trains, each 1 000 ft. long and weighing 3 000 lbs. per linear foot; two long-distance passenger trains, each 1 000 ft. long and weighing 1 500 lbs. per linear foot; six Rapid Transit electric trains, each 500 ft. long and weighing 1 200 lbs. per linear foot.

The surface cars, on the two tracks provided for them, should run at a speed of at least 15 miles per hour. They would, therefore, be twice as far apart as on the street approach; a distance of 100 ft. from center to center of cars would be closer than practicable. This distance, with cars weighing 40 000 lbs., gives a load of 400 lbs. per linear foot of track.

For the calculation of the cables, anchorages and the towers above the floor level, this load is equivalent to 8 421 lbs. per linear foot of bridge, covering the whole length of the main span. The writer, therefore, will assume a moving load of 8 500 lbs. per linear foot of bridge for these calculations.

For the calculation of the stiffening trusses, the loads on the surface-car tracks may be neglected, as they are nearly uniformly dis-

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tributed over the length of the bridge. The stresses produced by the trains 1 000 ft. long and those produced by the trains 500 ft. long would have to be calculated separately and then added, if the exact stresses corresponding to these loads are wanted. No equivalent load of one length will give the same stresses in every member of the stiffening trusses as the two loads of different lengths.

For the purpose of comparing the total weights of different designs, an equivalent load, giving the same average stresses and total weights as the two loads of different lengths, may be found.

In a span of 2 800 ft., from center to center of end hinges, a train 500 ft. long will give, on an average, stresses about 64% as large as those produced by a train 1 000 ft. long of the same weight per linear foot. The equivalent load of 1 000 ft. length for the calculation of the stiffening trusses, therefore, is 13 600 lbs. per linear foot of bridge. This load cannot be used for finding the correct stresses in the different truss members, but only for finding the total weights of the trusses.

For a bridge of 2 800 ft. span between the end-hinges of the stiffening trusses the total dead load of the main span, inclusive of the cables and suspenders, and 500 lbs. per linear foot of bridge for pipes, etc., is 28 000 lbs. per linear foot. This is the result of an exact estimate based on an actual design with deep three-hinged stiffening trusses.

With these data it is now possible to determine the deformations of the bridge caused by the moving loads and changes of temperature, and to estimate the importance of the deformations in selecting the design.

The bridge here considered has a span of 2 850 ft. from center to center of towers. We will first consider a design with deep, three-hinged stiffening trusses, and with the shore spans supported on piers.

The rear cables are 1 395 and 1 851 ft. long. The dip of the cables of the main span is one-eighth of the span from center to center of the towers. The ends of the cables are 500 ft. lower than their highest points at the towers. The drop of the cables at the center of the main span, due to a rise of temperature of 120°, is 7.4 ft. If it be assumed that wire having an elastic limit of 170 000 lbs., and an ultimate strength of at least 200 000 lbs. per square inch, is used in the cables; and if

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the cables are properly designed, so that the bending stresses in them are small and well known, a unit stress, for dead and live load, of 70 000 lbs. per square inch may be taken. The unit stress from live load alone will then be $\frac{8\,500}{36\,500} \times 70\,000 = 16\,300$ lbs. per square inch. The rear spans of the cables are 1 740 and 1 260 ft. long. The weight of the rear cables is 6 400 lbs. per linear foot of bridge. From these data the drop of the cables at the center of the main span due to the moving load has been calculated. It is 6.4 ft. The total variation in the elevation of the cables at the center of the main span, therefore, is 13.8 ft.

If the tracks are level in the lowest position, the grade in the highest position of the main span will be practically uniform, and equal to 1%, rising from the towers to the center of the span. It is assumed that the stiffening trusses have three hinges. If they are 140 ft. deep in the center of the half-spans and 60 ft. deep at the towers and the center of the span, then the upward bending of each half-span for the highest position of the main span is only $\frac{3}{4}$ in. above the curve for the mean load and temperature. The stiffening trusses of each half-span remain practically straight for uniform moving loads of any amount and for all temperatures.

If the stiffening trusses have only two end hinges and no center hinge, and if the tracks are level in the lowest position of the main span, then the whole span will deflect approximately in the arc of a circle. The grade at the towers will be 2% in the highest position of the main span, and will gradually decrease to nothing at the center of the main span. If the stiffening trusses were fixed at the towers and without a center hinge, the maximum grade would be 2%, but it would occur near the center of each half-span.

Three-hinged, deep, stiffening trusses, therefore, have the advantage that the maximum grade is only one-half of that which occurs if there are fewer than three hinges.

The other principal factors influencing the maximum grade are the dip of the cables in the main span and the length of the back spans of the cables. The shorter the back spans of the cables and the larger their dip in the main span, within practical limits, the less is the maximum grade. The maximum grade can be reduced in all cases by building the tracks with a sag in their lowest position.

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The use of three-hinged, deep, stiffening trusses for railroad suspension bridges, however, has other and more important advantages. They consist mainly in their greater economy, in smaller secondary stresses due to the deformation, and in closer agreement between the actual and calculated stresses. To prove this it is necessary to calculate the stresses due to the dead and moving loads, the changes of temperature, and the wind pressure in suspension bridges with stiffening trusses of three, two or no hinges.

Stresses Produced by the Change in Length of the Cables in Suspended Stiffening Trusses with Two End-Hinges.

This change in length of the cables and of their consequent elevation in the main span is due to changes of temperature and moving load. To calculate the stresses produced, the sections of the stiffening trusses should be known. It will be assumed that the stresses per square inch in the chords of the stiffening trusses, produced by their rise and fall with the cables, are uniform over the whole length of the trusses. This is more nearly true in properly designed stiffening trusses than the frequent assumption that the moments of inertia of the stiffening trusses are uniform over their whole length. The writer will neglect the influence of the deformation of the web members, which is small in the shallow trusses here considered. In this case the trusses will deflect in the arc of a circle. If δ is the rise of the cables at the center of the main span, if the stiffening trusses are assumed to be straight before the rise, they will, after the rise, be arcs of circles with a radius $= R$. If the span between the end hinges is l, then $R = \frac{l^2}{8\,\delta}$. If d is the depth of the stiffening trusses, and t is the stress per square inch in their chords produced by their bending, then $t = \frac{4\,\varepsilon\,\delta\,d}{l^2}$, where ε is the modulus of elasticity of the chords. This is, on account of splices and tie-plates, somewhat larger than that of steel, and may be taken at 30 000 000 lbs. per square inch of gross section. If special values for d, δ, and l are introduced into this formula, as follows: $d = 60$ ft.; $l = 2\,800$ ft., $\delta = 6.9$ ft.; then $t = \pm 6\,337$ lbs. If d is taken equal to 140 ft., then $t = \pm 14\,786$ lbs. The δ used above is one-half the total variation in the elevation of the cables at the center of the main span. If the suspenders are so adjusted that there are no stresses in the stiffening trusses at the mean

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temperature and with one-half of the largest uniform moving load on the bridge, then the stresses produced by the variation in the elevation of the cables are the smallest possible. They are then of opposite sign and equal for the two extremes of temperature and moving load.

It is not so important, however, to find the largest amount of these stresses due to the variation in the elevation of the cables as to find those stresses which coincide with the largest stresses from the moving load. The variation in the moving-load stress in the cables which occurs at the same time with maximum stresses in the stiffening trusses due to the moving load is about 61% of 6.4 ft. = 3.9 ft. The corresponding δ is $= \frac{1}{2}(7.4 + 3.9) = 5.65$ ft.

The values of t for stiffening trusses 60 ft. and 140 ft. deep, which must be considered in obtaining the total maximum stresses in the stiffening trusses are, therefore, $t_{60} = \pm$ 5 190 lbs.; $t_{140} = \pm$ 12 110 lbs.

Stresses Produced by the Change in Length of the Cables in Suspended Stiffening Trusses of Three Hinges.

The writer has published* methods for calculating these stresses, and will use the same formulas herein: $T = \frac{\delta_1 (w + q) L^2}{h l^2}$; where T is the load per linear foot of bridge which will produce on a span, of the dimensions of the half-span of the stiffening trusses, and freely supported at both ends, stresses equal to the stresses produced by the variation in the length of the cables occurring at the same time with the maximum moving-load stresses in the stiffening trusses. In three-hinged stiffening trusses those stresses which are due to the lengthening of the cables from moving load are of opposite sign to the maximum moving-load stresses occurring at the same time; their neglect is therefore on the safe side, giving maximum stresses considerably too large. The δ_1 in the above formula is therefore one-half the change in the elevation of the cables at the center of the main span due to changes of temperature only.

$\delta_1 = 3.7$ ft.; $w + q = 34\,000$; $L = 2\,850$; $l = 2\,800$; $h = \frac{L}{8}$.

$w + q$ is intentionally taken less than the maximum value, for the purpose of partly neutralizing the error arising from the neglect of the stretch of the cables by the moving load, which produces negative stresses.

* *Engineering News*, November 7th and December 12th, 1901.

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If these values are introduced in the formula for T we obtain $T = 366$ lbs. The stiffening trusses are 140 ft. deep at the center of the half-spans. We obtain, therefore, for the stress in each chord of the stiffening trusses at the center of the half-span, $\pm$ 320 400 lbs. The formula used neglects the deformation of the stiffening trusses produced by these stresses. This deformation has the effect of reducing the above stress by about 6 per cent. It is, therefore, $\pm$ 301 000 lbs. The cross-section of the chords of the bridge, here considered, is slightly different for the top and bottom chords. It averages 350 sq. ins. at the center of the half-span. The stress per square inch, therefore, is $\frac{301\,000}{350} = 860$ lbs. It was found above that the corresponding stress for stiffening trusses 140 and 60 ft. deep, without any center hinges, was $\pm$ 12 110 and $\pm$ 5 190 lbs. per square inch of cross-section.

The stresses in the chords of three-hinged stiffening trusses produced by changes in the length of the cables vary with change in the depth of these trusses in the same ratio as the stresses due to an unequally distributed moving load. The chord sections must vary nearly as the sum of these stresses. The former stresses per square inch of chord section, therefore, are nearly the same, whatever the depth of the three-hinged trusses. For two-hinged stiffening trusses, 60 ft. deep, the stresses due to the change in the elevation of the cables, therefore, are about six times as large as in three-hinged trusses of the same depth.

In stiffening trusses, continuous or fixed at the towers, the temperature stresses are still larger than in those with two hinges. Any disadvantage arising from excessive temperature stresses in two-hinged trusses, therefore, is much increased by the omission of the end hinges.

Wind Stresses in Two-Hinged Suspended Stiffening Trusses.

For estimating the importance and amount of these wind stresses, we take again a span of 2 850 ft. from center to center of towers, with the dip of the cables equal to one-eighth of the span. The stiffening trusses are 60 ft. deep, and have a span of 2 800 ft. between centers of end hinges. The width from center to center of trusses is 92 ft. The wind pressure is 1 600 lbs. per linear foot of bridge. This corresponds to about 30 lbs. per square foot of exposed surface. If the top and bottom chords have equal sections, and if there is a substan-

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tial vibration bracing, preventing any considerable deformation of the cross-section, the lateral deflection of the top and bottom lateral systems must be nearly alike. In this case the wind pressures resisted by the two lateral systems are also nearly alike. If the stiffening trusses alone would resist the wind pressure, we would obtain stresses of $\pm$ 4 261 tons at the center of the span in each of the four chords. The cables, however, assist in resisting the wind pressures. To estimate the amount of this assistance, the horizontal deflection must be calculated. For this purpose it is necessary to assume the chord sections of the stiffening trusses and the dead loads. The gross section of the chords of the stiffening trusses will be assumed to be 700 sq. ins. throughout their length. In an actual design these sections would decrease in the end thirds of the span. The bridge here assumed, therefore, will have the same lateral deflection as a bridge with somewhat larger maximum chord section of the stiffening trusses. The dead load for this design is about 38 000 lbs. per linear foot, the moving load is as given before. The wind stresses in the stiffening trusses are largest when the moving load on the cables is small. Those wind stresses in the chords of the stiffening trusses are wanted which occur together with the largest stresses from the moving load. Some of the largest stresses from the moving load occur when the total load, w, on the cables is as little as 42 000 lbs. per linear foot. If the lateral deflection of the trusses at the center of the span is δ, the lateral deflection of the cables is the same amount. If the cables are not cradled, and if the horizontal force acting on them is uniform per linear foot of bridge, then the horizontal projection of the cables is a parabola. The tangents to this parabola at the saddles form angles, α, with the axis of the bridge, given by the equation tan. $\alpha = \frac{2\,\delta_1}{L} = \frac{4\,\delta_1}{L}$, where δ_1 is the lateral deflection at the center of the span, and L is the span from center to center of towers. The horizontal component of the tension in the cables, which have a dip of one-eighth of the span, is approximately $w\,L$. The lateral pressure on each tower, due to the lateral inclination of the cables in the main span, therefore, is approximately $w\,L\,\frac{4\,\delta}{L} = 4\,w\,\delta_1$.

The lateral pressure exerted on the two towers is $8\,w\,\delta_1$. The horizontal pressure, x, per linear foot of bridge on the cables must therefore be $x = \frac{8\,w\,\delta_1}{L}$.

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If the horizontal force pushing the cables acts at the center of the span only, the horizontal projection of the cables will consist of two straight lines forming angles, β, with the axis of the bridge. The angle β is given by tan. $\beta = \frac{2\,\delta}{L^2}$, and the horizontal force which, if acting on the cables at the center of the span produces there a lateral deflection of δ_2, is $y = 4\,w\,\delta_2$, or one-half the horizontal force which, uniformly distributed over the length of the span, produces the same lateral deflection of the cables.

The horizontal force acting on the cables is partly the wind pressure on the exposed surface of the cables and on half the surface of the suspenders, and partly the horizontal component of the laterally inclined pull of the suspenders. The lateral inclination of the suspenders is small near the ends of the span where they are long. The total lateral pressure on the cables, therefore, is a nearly uniform pressure per linear foot of bridge plus a pressure acting near the center of the span.

The writer will assume, as a sufficient approximation to the facts, that the horizontal wind pressure acting on the cables is 200 lbs. per linear foot of bridge and a pressure acting at the center of the span, large enough to produce, with the uniform pressure, the same lateral deflection as that of the stiffening trusses.

The lateral forces acting on the stiffening trusses are, therefore, a uniform pressure of 1 400 lbs. per linear foot of bridge and a pressure acting against the wind at the center of the span. If the latter is y, the lateral deflection of the cables is obtained from the two equations: $x = \frac{8\,w\,\delta}{L}$, and $y = 4\,w\,\delta_2$; where $x = 200$, $L = 2\,850$, $\delta = \delta_1 + \delta_2$, $w = 42\,000$ lbs.

We obtain $\delta = \frac{2\,850 \times 200}{8 \times 42\,000} + \frac{y}{4 \times 42\,000}$, or

$$\delta = 1.697 + \frac{y}{168\,000}.$$

If we neglect the effect of the deformation of the lateral diagonals and struts we find the lateral deflection of the stiffening trusses from the formula:

$$\delta = \frac{1\,400 \times 2\,800^4}{76.8 \times 30\,000\,000 \times 700 \times 92^2} - \frac{y\,2\,800^3}{48 \times 30\,000\,000 \times 700 \times 92^2}$$

$$\delta = 6.303 - \frac{y}{388\,700}.$$

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From the foregoing equations,

$$1.697 + \frac{y}{168\,000} = 6.303 - \frac{y}{388\,700}, \text{ or } 4{,}606 = \frac{y}{168\,000} + \frac{y}{388\,700}$$

From this, $y = 540\,200$ lbs., and $\delta = 4.913$ ft.

The average moment of wind pressure to be resisted by the stiffening trusses, if the cables would not assist in resisting the wind-pressures, would be $\frac{2}{3} \times \frac{1\,600 \times 2\,800^2}{8 \times 2\,000} = 522\,667$ foot-tons.

In consequence of the assistance of the cables in resisting the wind pressure, this average moment is reduced to 362 800 foot-tons; or the average moment from wind pressure in the stiffening trusses is reduced to that which would be produced by a wind pressure of 1 110 lbs. per linear foot of bridge acting on them.

The average shear in the lateral systems is reduced in nearly the same proportion by the assistance of the cables in resisting the wind pressure. For the purpose of making comparative estimates it can be assumed that, of the wind pressure of 1 600 lbs. per linear foot of bridge, 490 lbs. is carried by the cables and only 1 110 lbs. by the two lateral systems of the stiffening trusses.

Wind Stresses in Three-Hinged Suspended Stiffening Trusses.

For a bridge of the great weight of the Hudson River Bridge the center connection between the two half-spans of the stiffening trusses ought to be so designed that it is able to transfer shears in a horizontal and vertical direction, but no moments.

In this case each half-span acts as an independent span to resist wind pressures. The wind pressures carried by the lateral systems of the stiffening trusses to the center of the span are carried by the cables to the towers. The stiffening trusses, under extreme wind pressures, will deflect considerably at the center of the span. Special provision has to be made in the tracks to avoid a sharp angle in the rails. Designs for the center connection of the stiffening trusses and for the special rails at the center and ends of the main span have been made, submitted to eminent engineers, and found satisfactory. The uniform wind pressure carried by the cables is less than in the previous case, say 150 lbs. per linear foot. The trusses are higher, but the cross-sections of their members are very much smaller, and their width is less. The total area exposed to wind pressure, there-

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fore, is about the same as with two-hinged shallow trusses. The wind pressure acting on the trusses, therefore, is 1 450 lbs. per linear foot of bridge.

The wind stress in each chord, at the center of the half-span, therefore, is $\frac{0.725 \times 1\,400^2}{2 \times 8 \times 92} = \pm 906$ tons.

The average wind stress in the chords is approximately $\pm$ 610 tons. This is 31% of the average wind stress in the chords of two-hinged stiffening trusses.

The average wind stress in the lateral web system is 66% of that with two-hinged trusses.

Moving-Load Stresses in Two-Hinged and Three-Hinged Suspended Stiffening Trusses.

The maximum bending moment due to the moving load (neglecting the effect of the change in the length of the cables) in two hinged suspended stiffening trusses is $\frac{w\,l^2}{8}$ 0.1482, and in three-hinged suspended stiffening trusses is $\frac{w\,l^2}{8}$ 0.1506, where w is the load per linear foot of truss and l is the span. The maximum moments extend over greater lengths of two-hinged than three-hinged trusses.

The average bending moment in two-hinged trusses is $\frac{w\,l^2}{8}$ 0.113, or 76.25% of the maximum moment. In three-hinged trusses the average bending moment is $\frac{w\,l^2}{8}$ 0.09744, or 64.7% of the maximum moment. The average moment of three-hinged trusses is only 0.862 of the average moment of two-hinged trusses. These values of the averages are taken from a calculation made by Edwin Duryea, Jr., M. Am. Soc. C. E. The moments were calculated for 50 panel points in the half-span. These calculations, however, were made with a moving load of such length as to give the largest stress. They are approximately true for positive moments if the moving load is limited to 1 000 ft. in length. The negative moments will not be used in the calculation of the sections.

As will be shown later, the three-hinged stiffening trusses, 140 ft. deep, must distribute about 12 000 lbs. of 13 600 lbs. of moving load per linear foot of bridge; the remainder is distributed by the cables.

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The two-hinged trusses, 60 ft. deep, must distribute only seven-eighths as much, the remainder being distributed by the cables.

The largest chord stress from moving load is 3 170 tons in the three-hinged trusses and 6 296 tons in the two-hinged trusses.

Unit Stresses and Chord Sections in Two-Hinged and Three-Hinged Trusses.

The writer will assume the use of steel of from 80 000 to 90 000 lbs. ultimate strength, and 45 000 lbs. elastic limit, and will calculate the sections by using, of the two opposite maximum stresses, only that one which requires the largest section. The unit stress for tension per square inch of net section will be taken at 22 500 lbs., if wind stresses are omitted, and 20% more if they are added. The unit stress in compression will be taken at $22500 - 90\frac{l}{r}$, if wind stresses are omitted, and 20% more if they are added. The reasons for the selection of these values will be given later.

From the preceding investigations we obtain, for two-hinged trusses:

The largest chord stress from moving load, 6 296 tons;

The largest chord stress from wind pressure, $4\,261 \times \frac{1110}{1600} = \pm\ 2\,956$ tons;

The stress per square inch in the chords, from the change in length of the cables, $\pm$ 5 190 lbs.

For three-hinged trusses:

The largest chord stress from moving load, 3 170 tons;

The largest chord stress from wind pressure, $\pm$ 906 tons;

The stress per square inch in the chords, from the change in length of the cables, $\pm$ 860 lbs.

The $\frac{l}{r}$ for two-hinged trusses is 25. The $\frac{l}{r}$ for three-hinged trusses is 30. The net section is seven-eighths of the gross section.

This gives the largest top chord section for two-hinged trusses = 969 sq. ins.; for three-hinged trusses, 356 sq. ins.; the largest bottom chord section for two-hinged trusses, 970 sq. ins.; for three-hinged trusses, 356 sq. ins.

In two-hinged trusses it was found that the average chord stress from moving load is 76.25% of the maximum chord stress.

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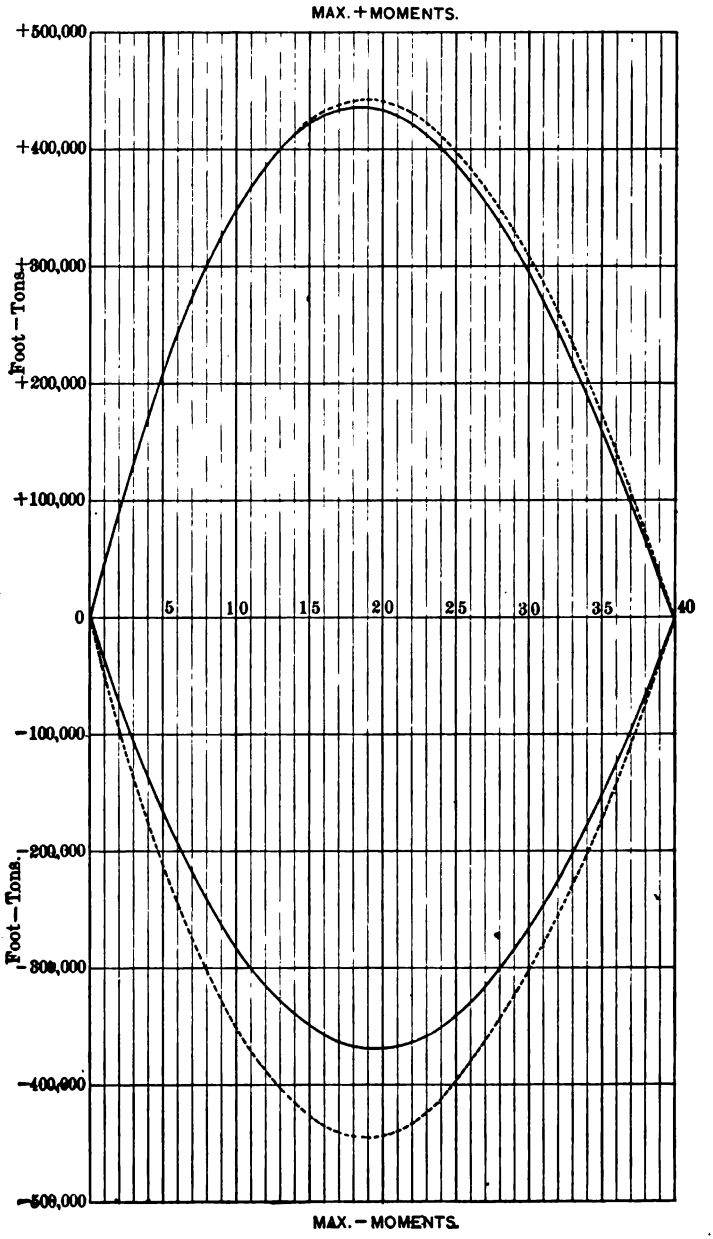

Fig. 1.

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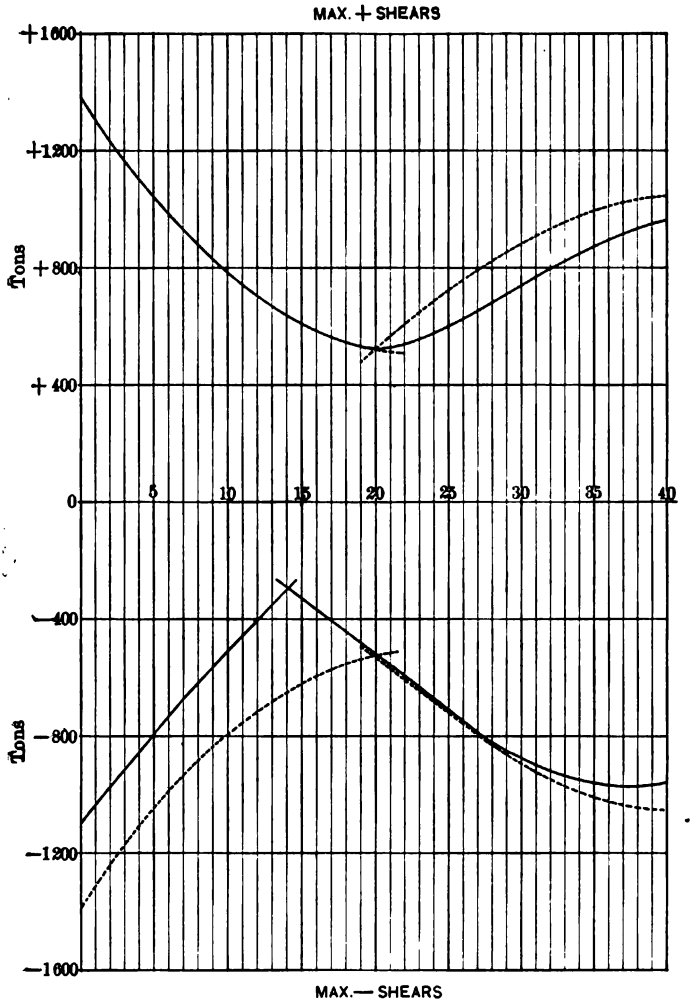

In Figures 1 and 2 the full lines give Maximum Moments and Shears due to a continuous moving Load 1000 ft. or less in length, the dotted lines, those due to an unlimited length. These diagrams, (Figs. 1 and 2) are for a moving load of 8 tons per foot per truss, and for a one-half span of 1400 ft.=40 panels at 35 ft.

FIG. 2.

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The average chord stress from wind pressure, according to these assumptions, is two-thirds of the maximum. The average chord stress due to change in length of the cables is three-fourths of the maximum.

The average chord section of two-hinged trusses, therefore, is approximately three-fourths of the maximum chord section, if allowance is made for the unavoidable surplus section near the ends of the span.

For three-hinged trusses the average chord section was obtained from a completed stress sheet, and was found to be three-fourths of the maximum chord section.

The top chords of the three-hinged trusses are slightly longer than those of the two-hinged trusses; allowing for the increase in weight caused thereby by increasing the chord section of the three-hinged trusses, we obtain an average chord section of 727 sq. ins. for the two-hinged, and 268 sq. ins. for the three-hinged, trusses. The difference is 459 sq. ins. Allowing 28% for connections, latticing and rivets, this means that the chords of the two-hinged trusses weigh 8 000 lbs. per linear foot of bridge more than those of the three-hinged trusses.

In this investigation those moments which produce compression in the top chords are called positive moments; those shears which produce tension in the diagonals rising toward the rear end of the span are called positive shears.

For a limited length of moving load the positive moments and shears are larger than the negative ones. For three-hinged trusses and an unlimited length of load, the positive and negative moments and shears are alike if two trains are used on each track. But with only one train the negative shears are smaller in some cases. Figs. 1 and 2 give the maximum moments and shears in three-hinged stiffening trusses due to a load 1 000 ft. long, or less, and those due to a load of unlimited length. The calculations were made by Mr. Duryea, who kindly offered them for the writer's use.

Before the webs and laterals of three-hinged and two-hinged trusses can be compared, another investigation is required, which will here be introduced.

Stresses in Lateral Systems Due to Moving Load.

No attention is generally given to these stresses, and experience with ordinary bridges seems to show that no harm results from this

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neglect. The proposed Hudson River Bridge, however, is so extremely different, in its general dimensions and in its number of tracks, from ordinary bridges, that it is not safe to conclude that these stresses may be neglected.

The writer will first consider the case of two-hinged stiffening trusses 60 ft. deep. A top and bottom lateral system and a substantial vibration bracing is assumed to be used.

The longitudinal center line will be called the axis of the bridge, and the line perpendicular to it at the center of the span, alone, will be called the center line.

If the tracks south of the axis, and west of the center line, and those north of the axis and east of the center line, are loaded, and the remainder unloaded, the west half of the south truss and the east half of the north truss will bend down, and the other parts of the trusses will bend up.

First, consider the vibration struts as being absent. The moving load on the south truss of the west half-span is two-thirds of the maximum moving load of this truss. The moving load on the north truss of the west half is one-third of the maximum moving load of this truss. The moving load on the south truss of the west half-span is different from that on the east half-span of the same truss by one-third of the maximum moving load per truss. The average stress in the chords is somewhat less than one-third of the maximum average moving-load stress. The maximum stresses from moving load in the chords are 6.5 tons per square inch. The average chord stress under the foregoing loading, therefore, is about 2 tons per square inch.

In the southwest truss there is compression in the top chords and tension in the bottom chords. In the northwest truss there is tension in the top chords, and compression in the bottom chords. The southwest truss, therefore, will deflect downward, the deflection at the quarter being $\delta = \frac{l^2 t}{4 \varepsilon d}$, where $t = 2$ tons, $\varepsilon = 15\,000$ tons, $d = 60$ ft., and $l = 1\,400$ ft. If these values are introduced we obtain $\delta = 1.09$ ft. The northwest truss will deflect upward an equal amount. If we consider the horizontal trusses formed by the top chords and top laterals and by the bottom chords and bottom laterals, we find that the top truss of the west half-span will bend northward, because the north chords expand and the south chords contract; the

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bottom truss will bend southward the same amount. The horizontal deflection at the quarter-span is $1.09 \times \frac{60}{92} = 0.71$ ft. In Fig. 3 the consequent deformation of the cross-section at the quarter-span is shown in dotted lines, but exaggerated.

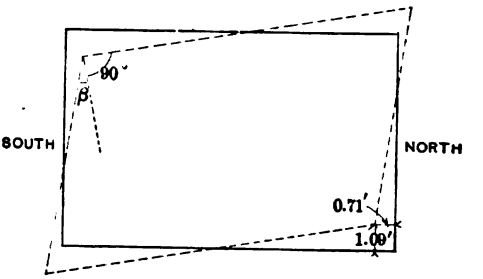

FIG. 3.

The vibration bracing and the rigid vertical posts practically prevent this deformation. By this vibration bracing a part of the weight on the south truss is carried over to the north truss and pulls this down and southward at the top. At the same time it pulls the south truss up and northward at the bottom. If the load per linear foot which is carried to the north truss is q, then the horizontal pull created in both lateral systems is $q \frac{92}{60}$. It will be assumed that the load q is uniform over the length of the half-span. The same chords serve for the horizontal and vertical trusses. The loads on the horizontal and vertical trusses are as 92 to 60. The depths are also as 92 to 60. The stresses produced by the horizontal and vertical forces, therefore, are equal, and the total stress in the chords is equal to twice the stress produced by the vertical forces alone.

The vertical deflection of each truss produced by the transference of the load q per linear foot from the south to the north truss, therefore, at the quarters, is $\delta = \frac{2\,q\,l^4}{76.8\,E\,I}$.

The average chord section is 727 sq. ins.; $l = 1\,400$ ft.; $E = 30\,000\,000$ lbs., and $I = 727 \times \frac{60^2}{2}$. If these values are introduced in the formula, then $\delta = \frac{q}{393}$. The deflection of the horizontal trusses, due to the deformation of the chords, is $\delta_1 = \delta \times \frac{60}{92} = \frac{q}{602}$.

The influence of the deformation of the lateral diagonals of the bottom lateral system cannot be neglected, as the stresses produced in them are quite large, and their sections moderate. If the average stress

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per square inch of gross section in these laterals, produced by the foregoing distribution of the moving load, is 8 000 lbs., then the horizontal deflection produced by their deformation is 0.38 ft. In the absence of vibration struts, the change in the angle between the vertical posts and the floor beams would be β, where tan. β is approximately given by the equation:

$$\text{tan. } \beta = \frac{1.09 \times 2}{92} + \frac{0.71 \times 2}{60} = 0.04736.$$

This deformation is corrected by the deflections caused by the load q, which is transferred from the south to the north truss. The writer will assume that the deflection caused by the deformation of the web of the top lateral system is the same as that caused by the deformation of the bottom lateral diagonals. The deformation of the floor beams, serving as bottom lateral struts, is neglected.

We obtain

$$\text{tan. } \beta = \frac{2\left(\frac{q}{602} + 0.38\right)}{60} + \frac{2q}{393 \times 92}.$$

If the value of tan. β, thus found, is introduced, $q = 313$. The horizontal force per linear foot acting on each lateral system, therefore, is $313 \times \frac{92}{60} = 480$ lbs.

This pressure acts in one half-span in one direction, and in the other half-span in the other direction. The stresses produced by this force in the laterals must be added to those produced by wind pressure. The average lateral stresses are increased thereby by about 43 per cent.

If the total tension per square inch, net, in the laterals, is made 30 000 lbs., then their stress per square inch, gross, from moving load, is about 8 000 lbs. on an average, as previously assumed. The neglect of these stresses would give dangerously weak laterals near the center of the span. They must also be considered in dimensioning the vertical posts and the vibration struts.

In the three-hinged stiffening trusses, 140 ft. deep at the center of the half span, the lateral pressure in each lateral system is increased on an average about 140 lbs. per linear foot, or 20% of the wind pressures.

The wind stresses in the laterals of the three-hinged trusses were found to be 66% of those in the laterals of the two-hinged trusses.

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After making the necessary additions of moving-load stresses it is found that the total stresses in the lateral systems of the three-hinged trusses are $66 \times \frac{120}{143} = 55.4\%$ of those in the lateral systems of the three-hinged trusses.

The laterals and vibration struts of the three-hinged trusses weigh 1 500 lbs., those of the two-hinged trusses weigh 2 000 lbs., per linear foot of bridge.

Comparison of Web Systems of Two-Hinged and Three-Hinged Trusses.

For the calculation of the web systems of two-hinged and three-hinged trusses we have no wind stresses in either. The average shears for two-hinged and three-hinged trusses were calculated by Mr. Duryea by using 50 panels in each half-span. The average shear was found to be 1% larger in three-hinged than in two-hinged trusses. The loads were taken of unlimited length. The assistance given by the cables was not taken into account. Allowing for the latter, the average shear will be only seven-eighths as large in the 60-ft. deep two-hinged as in the 140-ft. deep three-hinged trusses. The shears due to changes in the length of the cables are about six times as large in two-hinged as in three-hinged trusses. In the former they are about 5 200 lbs. per square inch. The shears of the three-hinged, but not those of the two-hinged, trusses are partly resisted by the top chords, as the latter are curved in the three-hinged, and straight in the two-hinged, trusses. The bending moments in the vertical posts, due to the transference of loads from one truss to the other for certain positions of the moving load, are about three and one-half times as large in two-hinged as in three-hinged trusses. Everything considered, the weights of the webs of both kinds of trusses are about alike, and are equal to 2 600 lbs. per linear foot of bridge.

The total weight of the three-hinged trusses is 8 800 lbs.; that of the two-hinged trusses is 8 500 lbs. more, or 17 300 lbs. per linear foot of bridge. The addition of 8 500 lbs. to the weight of the stiffening trusses, if two-hinged trusses are used, has the effect of increasing the weight of the cables about 1 875 lbs. per linear foot of bridge. The dead load of the bridge with two-hinged shallow stiffening trusses, therefore, is 38 375 lbs. per linear foot of bridge, or about as assumed in these calculations.

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The cables cost at least twice as much per pound as structural steel. The use of two-hinged instead of three-hinged trusses adds to the cost of the main span the same amount as the addition of 8 500 + 3 750 = 12 250 lbs. of structural steel per linear foot of bridge. It adds 28.4% to the total load per linear foot of main span, and increases the cost of the rear cables, anchorages, towers and tower foundations in nearly the same proportion.

The design of a suspension bridge requires a double calculation—first, an approximate one, to determine the general dimensions, and then an accurate one, based on the approximate knowledge of the final results. The calculation and comparison here made cannot claim accuracy, and would not be satisfactory for a design to be executed. It is exact enough, however, for this purpose, which is the comparison of different designs, with a view of selecting one of them for the final calculation.

Braced Chains Compared with Suspended Three-Hinged Stiffening Trusses.

There exist a number of short-span suspension bridges with braced chains. The same system has been proposed for a suspension bridge across the Hudson River, in New York City, the links of the chains to be made of steel wires wound around sleeves. The chains perform the function of the cables and of the chords of the stiffening trusses. The moving loads in the present case are little longer than one-third of the span. The same length of moving load produces the maximum tension and the maximum bending stresses in the trusses formed by the braced chains. A different position of the moving load, however, is required to produce the maximum tension and the maximum bending in a large part of these trusses. With suspended stiffening trusses it is necessary to provide in the trusses for the maximum bending produced by any position of the moving load, and in the cables for the maximum tension. With braced chains one must provide only for the largest sum of the bending stress and tension occurring at the same time. Cables have a uniform section throughout, though the stresses increase toward the towers. Chains can be made of variable section to suit the stresses. The chords of the stiffening trusses are made of structural steel. The braced chains here considered are proposed to be made of wire of three to four times the elastic strength of structural steel. The weight of the metal re-

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quired to resist the bending stresses, therefore, is much smaller in the latter than in the former.

Braced chains, however, have serious disadvantages. To prevent compression in the chains, hinges must be introduced, at the towers and anchorages, into the trusses formed by the braced chains. On account of the great stresses in the chains, ordinary hinges are impracticable. Toggle joints, therefore, have been proposed. These are very imperfect hinges, leaving large variable moments which increase the sections required in the chains. The breaking up of the cables into links of chains of about 50 ft. horizontal length, and the introduction of four toggle joints add more than one-third to the weight of the metal required for continuous cables of the same average section. All the connections between the links must be made of steel of only one-third to one-fourth of the strength of the wire of the chains. They are, therefore, very heavy.

The four chains would each have a depth of about 12 ft. This necessitates covers of about twice the weight of the covers of continuous cables for a bridge of the same span and capacity. The absence of hinges at the center of the main span and the presence of very imperfect hinges at its ends entail considerable stresses due to the change in length of the chains. Two suspended wind trusses are needed at the level of the two decks of the floor, to stiffen the platforms laterally. If these two trusses are connected by bracing in vertical planes (as has been proposed), so as to form also two vertical trusses, there will arise in them, in consequence of vertical deflections, large bending stresses due to the moving load and the changes of temperature.

An unequally distributed moving load produces in the chains, first, a tension equal to that which would be produced in unbraced cables; second, in one of the chains of each truss a compression, in the other an equal tension, due to a bending moment. The total tensions per square inch in the upper and lower chains, therefore, are different. If the lower truss is of the same depth as the upper one, then the arithmetical sum of the stresses per square inch in its two chords is the same as the difference between the stresses in the upper and lower chains above them. If the lower truss is shallower or deeper than the upper truss the stresses per square inch in it (due to moving load and changes of temperature) are smaller or larger than

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those above given, in proportion to the ratio of the heights of the lower and upper trusses.

The stresses per square inch in the lower trusses, produced by the rise and fall of the cables in the main span are the same for the same rise or fall of the braced or unbraced cables. They are proportional to the height of the lower trusses. It is evident that everything is in favor of shallow suspended wind trusses. A depth of 30 ft. from center to center of chords, therefore, should be adopted for these trusses. With the upper floor beams riveted between the posts, this allows sufficient clearance between the upper and lower decks.

Since these trusses serve the purpose of resisting the wind pressures and not the vertical forces, it would be still better to omit the diagonals between the top and bottom chords. In this case, there are no stresses from changes of temperature, and the stresses from moving load are greatly reduced. The weight of the platforms is sufficient to prevent any buckling of the chords in a vertical direction.

The large surface exposed to the wind by the deep chains and by the double set of trusses greatly increases the wind pressures. For each linear foot of bridge there will be about 80 sq. ft. exposed to the wind, giving a wind pressure of 2 400 lbs.

Calculation of the Suspended Wind Trusses.

For determining the amount of wind pressure transferred by the cables to the towers, the total load on the bridge must be known. The wind stresses in these trusses are largest when the load is smallest. But the stresses from moving load in the wind trusses are largest with a full moving load on the ten tracks on which we assumed uneven moving loads. The writer, therefore, will assume a moving load of 7 000 lbs. and a dead load of 33 000 lbs. per linear foot of bridge. This gives a total load of 40 000 lbs. per linear foot of bridge. The dip of the cables will be assumed at one-tenth of the span. There are various reasons why a smaller dip than one-eighth of the span is desirable with braced cables. The comparison of designs with different dips can alone show which is the best. One-tenth of the span is taken mainly because an eminent engineer, who has undoubtedly made many calculations, has chosen this dip for his design for the Hudson River Bridge. In this case the cables carry to the towers a wind pressure which is about uniform per linear foot of bridge. If x is the wind press-

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ure per linear foot of bridge carried by the cables, then $x = \frac{8\,H\,\delta}{L}$, where H is the horizontal pull in the cables, L the span, and δ the deflection at the center of the span. If the numerical values for these quantities are introduced into the formula, $x = 140.4\,\delta$.

The wind stress in the chords of the wind trusses will be nearly uniform throughout their length; it will be taken at 23 000 lbs. per square inch of net section in tension, and the same amount, reduced by a column formula, per square inch of gross section in compression. Of the two stresses, only that one which gives the larger section is to be used. The chords are supported every 50 ft. The formula $23\,000 - 92\,\frac{l}{r}$ is used, taking $\frac{l}{r} = 60$. Then the unit stress per square inch of gross section will be 8.75 tons. The lateral deflection of the wind truss, neglecting the effect of the deformation of the diagonals and struts, will be found from the formula, $\delta = \frac{l^2 t}{4\,\varepsilon\,d}$; where $t = 8.75$ tons, $\varepsilon = 15\,000$ tons, $l = 2\,800$, and $d = 92$ ft. We obtain $\delta = 12.4$ ft. From this, $x = 140.4\,\delta = 1\,740$ lbs. The lateral deflection of this bridge, under the load and wind pressure assumed, therefore, is about 12.4 ft. The cables carry a uniform wind pressure of 1 740 lbs. per linear foot of bridge to the towers. The wind trusses would have to be designed to carry a wind pressure of 660 lbs. per linear foot of bridge, which is to be considered as a moving load. In addition to these wind stresses there are stresses from moving load arising in the same manner as in a bridge with suspended stiffening trusses. These moving-load stresses increase the average shear in the laterals to about the amount that would be produced by a wind pressure of 960 lbs. per linear foot of bridge. The average moment in the chords is increased to about the amount that would be produced by a wind pressure of 810 lbs. per linear foot of bridge. These stresses from the moving load in the wind trusses could be avoided by making the connection of the floor beams with the trusses hinged, as has been done in a recent bridge across the Rhine, in Germany. This, however, would detract from the rigidity of the platform, therefore it will not be adopted here. If the foregoing equivalent pressures be assumed for calculating the weight of the wind trusses, the following is the chord stress in the center of the half span: $\frac{405 \times 2\,800^2}{8 \times 92 \times 2\,000} = \pm\,2\,157$ tons.

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If, for all stresses combined, there is taken a unit stress of 27 000 lbs. per square inch, net, in tension, and $27\,000 - 108\,\frac{l}{r}$ in compression, the maximum chord section is 210 sq. ins., gross. The average chord section would be $\frac{2}{3} \times 210 = 140$ sq. ins. If 33% is allowed for connections and surplus section near the ends of the span, the weight of the four chords becomes 2 530 lbs. per linear foot of bridge. Substantial vertical posts, with the upper floor beams riveted between them, would be required to keep the two decks vertically above each other. The laterals, inclusive of these posts, weigh about 970 lbs., and the whole wind trusses 3 500 lbs., per linear foot of bridge. Therefore, we have suspended wind trusses weighing 3 500 lbs., compared with suspended two-hinged stiffening trusses weighing 8 800 lbs., per linear foot of bridge. The floor, with 50-ft. panels, however, would be about 1 000 lbs. per linear foot of bridge heavier than the floor for the 35-ft. panels of a bridge with deep stiffening trusses.

The Stiffening Trusses Formed by the Braced Chains.

The upper stiffening trusses formed by the chains and their bracing, together with the suspenders, must now be compared with the cables and suspenders of a bridge with suspended three-hinged trusses. Before this comparison can be made, some other features of the design must be determined.

The shore spans may be supported either from piers below or from the rear cables. The former arrangement permits the deflection of the tracks to one side of the axis of the bridge immediately beyond the towers. This is necessary if the New York approach near the bridge is located west of Eleventh Avenue. If the shore spans are supported from the rear cables, the suspenders prevent the deflection of the tracks for a considerable distance beyond the towers. To come near to the surface at the terminal station, the New York approach should go some distance north or south before turning east. Land is cheaper west than east of Eleventh Avenue; this consideration may limit the choice between different designs. A bridge with two equal approach spans carried by braced cables would be more beautiful, and, therefore, offers a more interesting theoretical problem, though it may not be practicable. The writer, therefore, will assume two such approach spans, each 1 600 ft. long from the center of each tower

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to the face of each anchorage. To avoid excessive bending moments in the braced cables of the approach spans, when either the main span or an approach span alone is loaded, anchored bents will be assumed at the centers of these approach spans. These fix the elevation of the approach spans at their centers, and have the effect that the bending moments in the braced cables of the main and approach spans are of more nearly the same size. The cables can be put on rollers on the towers, or they may be attached to their tops. In the latter case the towers should be of moderate width, otherwise their weight is greatly increased by bending stresses due to changes of temperature and moving load. Broad towers have also the drawback that the span from center to center of towers is thereby increased, as the least clearance between the piers is given by law. Therefore, towers 30 ft. wide at the top and 80 ft. wide at the bottom will be assumed. In this case, the horizontal forces in the direction of the axis of the bridge are very moderate, and, for the purpose of comparison, may be neglected. The attempt to resist horizontal motion of the cables at the tops of the towers by means of broad towers attached to the cables adds much more to the weight of the towers than can be saved in the braced chains. The ends of the wire-link chains will be taken 100 ft. inside the anchorages and 360 ft. below their position at the towers. At the center of the rear spans the cables are 270 ft. lower than at the towers. The formulas derived by Dr. Th. Schäffer for the calculation of braced arches* will be used for calculating the stresses due to changes of temperature. The main difference between the main span of the bridge here considered and an arch is the motion of the ends of the main span of the bridge, while the hinges of an arch bridge are fixed in position. Another difference arises from the larger unit stresses and consequent greater deformations in a suspension bridge.

Change in Position of the Tops of the Towers, Due to Changes of Temperature.

It will be assumed that the wire chains are fixed 100 ft. from the face of the anchorage. The exact dimensions of the rear spans and the position of the supports in the anchorages and at the tops of the towers can be given only after a design has been made. For these calculations an ordinary hinge will be assumed, 100 ft. from the face

* Published in the "Handbuch der Ingenieurwissenschaften," Zweite Abtheilung.

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of the anchorage and on the towers in their center line. The relative position of the important points of the center line of the rear stiffening trusses is given in Fig. 4.

It will be assumed that the changes of temperature change the length of the steel members by $\frac{1}{1200}$.

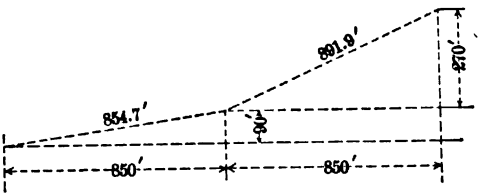

FIG. 4.

The intermediate bents of the shore spans are oscillating bents, allowing horizontal motion at the top. Their horizontal motion, due to changes of temperature, will be $\frac{854.7}{1200} \times \frac{854.7}{850} = 0.718$ ft. The horizontal motion at the tops of the towers, due to the expansion of the chains between the oscillating bents and the towers, is $\frac{891.9}{1200} \times \frac{891.9}{850}$ $= 0.78$ ft. The total horizontal motion of the tops of the towers, due to the change in length of the rear chains, is 1.498 ft.

The change in the height of the towers produces an opposite horizontal motion. If the steel towers are assumed to extend to 25 ft. above high water, they are about 460 ft. high to the hinge. The opposite horizontal motion at the top of tower, due to the change in height, is $\frac{460}{1200} \times \frac{270}{892} = 0.116$ ft. The total horizontal motion at the top of each tower, due to changes of temperature, therefore, is $1.498 - 0.116 = 1.382$ ft. The change of temperature changes the dip in the main span, and, in consequence, the tension in the chains, and their length; the towers and the suspended arch of the main span offer a resistance against motion of the tops of the towers. All these causes reduce slightly the horizontal motion of the tops of the towers. They will here be neglected.

TEMPERATURE STRESS AT THE CENTER OF THE MAIN SPAN; THE POSITION OF THE END HINGES FIXED.

The formula giving the stress per square inch at the crown of an arch is

$$\sigma = \pm \frac{15 E \alpha t \left(1 \pm \frac{2f}{h}\right)}{15 + \frac{32 f^2}{h^2}}, \quad \text{where } E = 28\,000\,000, \quad \alpha t = \frac{1}{2400},$$

$f = 285$ ft., and $h = 60$ ft.

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If these values are introduced, $\sigma_1 = \pm 2\,018$ lbs. and $\sigma_2 = \pm 2\,493$ lbs. The larger of the two stresses is in the inside chord of the arch. The formula holds approximately for a suspended arch.

Position of End Hinges Moving.—If the distance of the end hinges would change with the changes of temperature in the same proportion as the length of the steel members of the arch there would be no stresses from changes of temperature. The same stresses as those from changes of temperature would be produced by a change in the distance of the hinges by $\frac{1}{1200}$ of the span. This fact enables us to calculate the stresses produced by the expansion of the rear cables. This expansion changes the main span by $2 \times 1.382 = 2.764$ ft. One-twelve hundredth of the main span is 2.375 ft. The stresses produced by the change in length of the rear chains, therefore, are $\frac{2.764}{2.375} = 1.167$ times the temperature stresses, due to the change in length of the chains of the main span. The total temperature stresses, therefore, are $\pm 4\,367$ and $\pm 5\,394$ lbs. per square inch. These stresses occur at the center of the span, the larger ones in the upper chain. The results of the calculation are somewhat too large, because some effects of the deformation have been neglected.

The stresses which arise from those changes in the length of the chains which are due to a uniform moving load could be calculated in a similar manner. They amount to about 40% of those due to changes of temperature. This gives the total stresses due to the change in length of the chains by moving loads and changes of temperature of $\pm 6\,000$ and $\pm 7\,500$ lbs. per square inch. These results are somewhat larger than the actual stresses with perfect hinges. The imperfection of the hinges, however, will more than balance the slight error in the calculation.

These stresses decrease toward the ends of the main span, and would be zero at the hinges if these were perfect, which, however, is far from the case. The average temperature stress with perfect hinges would be little more than two-thirds of the maximum.

The calculation assumes such an adjustment of the web members of the suspended arches that the stresses here considered are zero at mean temperature and mean load. It is practically impossible to create this adjustment. Any deviation from it gives larger stresses than the

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calculated ones. In the formulas used, each of the four chains is assumed to have the same section through the whole length of the span, and the same section as each of the other three chains.

It is apparent from the results of the calculation of the stresses due to the change in length of the chains and from the essential similarity of the structures that the bending stresses from moving load in suspended two-hinged arches are approximately the same as in suspended two-hinged stiffening trusses of the same span and depth. For the purpose of a preliminary estimate, it will be assumed that they are the same. If the results show that an exact calculation must be made to determine the preponderance of advantage between this and other designs, the labor of calculating the exact stresses cannot be avoided by those who wish a first-hand knowledge of the subject. If the results show a very great advantage for other designs, this labor may be saved.

Approximate Calculation of the Braced Chains.

It is assumed that the maximum bending stress from moving load in each chain is 6 296 tons, the same as for two-hinged suspended stiffening trusses.

The direct tension in these chains, at the center of the span, is

$$\frac{41\,500 \times 2\,850^2}{8 \times 285 \times 2\,000} = 73\,920 \text{ tons.}$$

The average bending stress in the chains, due to the change in their length, is, at the center of the span, $\frac{1}{2}(6\,000 + 7\,500) = \pm 6\,750$ lbs. per square inch. Through the whole span the average is over $\pm 4\,500$ lbs. per square inch. The writer will first calculate the section required for the chains to resist the stresses of direct tension and those due to the change in length of the chains. The permissible unit stress in the chains is taken at 35 tons per square inch. The stress due to the change in length of the chains averages 2.25 tons per square inch. This stress, as far as it arises from moving load, does not everywhere coincide with the other maximum stresses due to moving load. To allow for this fact, only $\frac{7}{8}$ of 2.25 tons, or 1.93 tons, will be taken into consideration. This leaves for the direct tension a unit stress of 33.07 tons per square inch. With this unit stress the section required for the four chains at the center of the span is 2 235 sq. ins. At the ends of the span, the section required is 2 408 sq. ins. The average

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section required is 2 302 sq. ins. The largest bending stress is 6 296 tons in each chain. To make a low estimate for the weight of the chains, and to allow for the fact that the maximum bending stresses do not everywhere coincide with the other maximum stresses, the average bending stress will be taken at three-eighths of the maximum (in the two-hinged suspended trusses the average bending stress is 76% of the maximum), or $\frac{3}{8} \times 6\,296 = 2\,361$ tons. This, with a unit stress of 35 tons, gives 67.5 sq. ins. for each chain, or 270 sq. ins. for the four chains. The total average section of the four chains, therefore, is $2\,302 + 270 = 2\,572$ sq. ins. Allowing 33% for the weight of the connections, and 4% for the weight of the shell, and considering that the chains of the main span are 2.67% longer than the span, the weight per linear foot of bridge of the chains of the main span is $2\,572 \times \frac{10.2}{3} \times 1.33 \times 1.04 \times 1.0267 = 12\,420$ lbs. Of this weight, 1 304 lbs. per linear foot of bridge is required to resist the last-mentioned bending stresses in the chains. These latter are the only stresses in which there may be a considerable error; they are probably assumed too small, however, rather than too large.

The wire-rope suspenders carrying the platform have larger stresses in this design than in a design with suspended stiffening trusses. In the braced-chain design the suspenders attached to one floor beam must carry the maximum floor-beam load. In the design with suspended stiffening trusses the trusses distribute the load nearly uniformly over the whole span. The load to be carried by the suspenders in the former case is equivalent to 40 000 lbs.; in the latter, it is equivalent to 30 000 lbs. per linear foot of bridge. The allowance for impact must also be larger in the former than in the latter case. The weight of the suspenders with their connections is estimated in the former case at 700 lbs.; in the latter, though the suspenders are longer, at 550 lbs. per linear foot of bridge.

The webs between the chains have stresses similar to those in the webs of two-hinged suspended stiffening trusses. The former, however, must consist of a double system of adjustable tension diagonals, while the latter would consist of riveted members resisting tension and compression. The diagonals of the former must be either soft or medium steel, while the latter can be made of considerably stronger steel with higher unit stresses. The weight of the webs between the

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chains is approximately 3 000 lbs. per linear foot of bridge, against 2 600 lbs. for the weight of the webs of two-hinged suspended trusses.

The comparison can now be finished. It has been found that the weight carried by the suspenders is 4 300 lbs. per linear foot of bridge less in the braced-chain design than in the design with three-hinged suspended stiffening trusses. The chains and suspenders in the former design weigh 13 120, in the latter the cables and suspenders weigh 6 600, lbs. per linear foot of bridge. Allowing for the webs between the chains, the former design weighs $13\,120 + 3\,000 - 6\,600 - 4\,300 = 5\,220$ lbs. per linear foot of bridge more than the latter.

To fully appreciate the difference between the two designs here considered, the fact must be kept in mind that the cables, suspenders, wire chains and their connections cost about twice as much as other structural steel. This makes the difference between the two designs for the main span equivalent to a difference of 11 740 lbs. of structural steel per linear foot of bridge.

The unit stresses in wire chains or cables and in the structural steel of the stiffening trusses (as far as this is of the same kind) have been taken as the same in all the designs. In view of the practical impossibility of calculating accurately the braced-chain design, and of adjusting it as assumed in the calculations, the unit stresses ought to be taken at least 5% less in this design than in the design with three-hinged suspended trusses, if the same degree of safety is aimed at.

This would make the difference in economy between the two designs still larger.

The calculation of more exact stresses in the braced chains would be a work of months. In view of the great difference in the cost of the designs considered, and of the small error which might be found in the results of this approximate calculation, this work would be an unwarranted waste of labor.

Only the main span has here been considered. Any increase in the weight of the cables will affect the rear spans also. Any increase in the weight of the main span will affect the anchorages, towers and tower foundations.

The advisability of suspending the rear spans from the cables depends so much upon the local circumstances governing the cost of piers that it cannot well be discussed in a general way. An actual

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design of the shore spans of the Hudson River Bridge showed a great preponderance of advantage for spans supported from below.

If the rear spans are supported from the cables, the dead load only can be so supported. Most of the moving load coming on the rear span must be carried by it, if the main span is at the same time unloaded. If the whole dead load of the rear spans is carried by the cables, they must be anchored down at both ends, as the moving load on the main span will otherwise lift them up. A rear span equal in length to half the main span will be subject to stresses about twice as large as those in the stiffening trusses of the main span. Therefore, in most cases it becomes advisable to anchor the rear spans by bents near the center, so as to reduce the otherwise excessive stresses. The cases where the rear spans can with advantage be suspended from the cables are very exceptional.

Therefore, in what follows, the writer will give methods for the exact calculation of a suspension bridge with three-hinged, deep stiffening trusses. The rear cables will be assumed to carry only their own weight, the rear spans being supported on piers.

The Calculation of the Stresses from Moving Load, in Three-Hinged Stiffening Trusses.

In the following it will first be assumed that the curve of the cable is, and remains under all circumstances, a parabola, then the tension in the wire-rope suspenders supporting the stiffening trusses, though variable for different kinds of loading, will always be uniform over the whole length of the span. After this, there will be considered separately the modifications produced by the facts; first, that the stiffening truss is flexible and deflects under the influence of moments and shears, which produces a similar deformation of the cables, having the effect that the stresses in the rope suspenders become different throughout the length of the span; second, that the cables change their length under the influence of changing stresses and temperatures, which causes a drop or rise at the center hinge. This has the effect that the curve of the cable, instead of being one parabola, consists of two parabolas which meet at the center hinge at an angle. The cable, instead of having a horizontal tangent at the center, has then two tangents at the center rising or falling against each other. The former of these two cases, which would occur in winter with a

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small load on the bridge, if the suspenders in the center could take compression as well as tension, is but slightly modified by the fact that the suspenders are wire ropes.

1. Calculation of Stresses in Stiffening Truss, Neglecting its Elasticity and the Rise or Drop in the Center of the Span.—If a single load, P, at

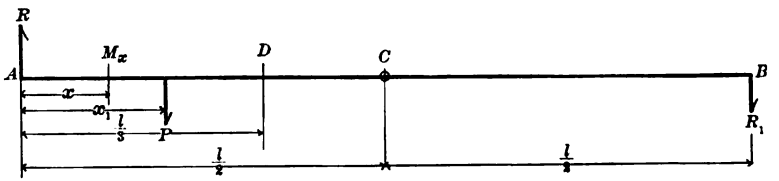

FIG. 5.

the distance, x_1, from the left end of the span (Fig. 5) is applied, the following equations of equilibrium obtain:

1st. The sum of all the vertical forces acting on the truss equals zero.

2d. The moment, in regard to the center hinge, of the forces acting on the left half-truss equals zero.

3d. The moment, in regard to the center hinge, of the forces acting on the right half-truss equals zero.

If the end reactions due to the load, P, are called R and R_1 (Fig. 5), if the suspender pull per linear foot of bridge produced by the load, P, is p, and if the span is l, we have:

$$R + l\,p - R_1 - P = 0 \quad \ldots\ldots (1)$$

$$R_1 \frac{l}{2} - \frac{p\,l^2}{8} = 0 \quad \ldots\ldots (2)$$

$$R \frac{l}{2} + \frac{p\,l^2}{8} - P\left(\frac{l}{2} - x\right) = 0 \quad \ldots\ldots (3)$$

From these we obtain

$$R = P\left(1 - \frac{3\,x_1}{l}\right) \quad \ldots\ldots (4)$$

$$R_1 = \frac{P\,x_1}{l} \quad \ldots\ldots (5)$$

$$p = P\,\frac{4\,x_1}{l^2} \quad \ldots\ldots (6)$$

The moment at the distance x from the left end is, for $x < x_1$

$$M_x = R\,x + \frac{p\,x^2}{2} = P\,x\left[1 + (2\,x - 3\,l)\,\frac{x_1}{l^2}\right] \quad \ldots\ldots (7)$$

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From this it follows that a load, P, between A and D (Fig. 5), produces a positive moment at all points between A and D, if $A\,D \leqq \frac{l}{3}$.

A load located in the third sixth of the span, counting from the left end, produces a moment 0 at a point at the distance x from the left end of the span, if $x_1 = \frac{l^2}{3\,l - 2\,x}$, as follows from Equation (7) by letting $M_x = 0$. At a smaller distance from the end it produces a negative moment, at a larger distance a positive moment. If $x_1 > \frac{l^2}{3\,l - 2\,x}$, then the moment at x is negative; if $x_1 < \frac{l^2}{3\,l - 2\,x}$, then the moment at x is positive.

If $x > x_1$, then $M_x = P\,x_1\left[\frac{x}{l^2}(2\,x - 3\,l) + 1\right]$

This equation indicates that M_x is equal to the ordinate of a parabola which passes the axis of x, for $x = \frac{l}{2}$ and $x = l$.

A force, P, in the left half-span produces positive moments between itself and the center of the span, and negative moments in the whole right half-span. Fig. 6 gives the moment curves produced by a load, P, in different positions on the span.

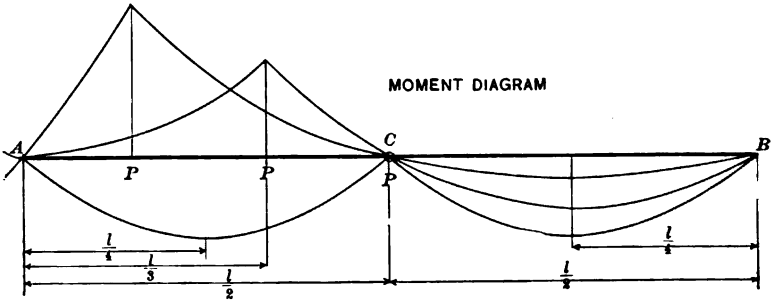

FIG. 6.

Shears.—The shear in a section distant x produced by a load, P, distant x_1 from the left end of the span, where $x < x_1 < \frac{l}{2}$, is $S = R + p\,x$, or $S = P\left[1 + \frac{x_1}{l^2}(4\,x - 3\,l)\right]$

This value of S is positive for all x, if $x_1 < \frac{l}{3}$. If $\frac{l}{3} < x_1 < \frac{l}{2}$, then the load, P, produces a shear, 0, at a section distant x from the

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left end of the span; when $x_1 = \frac{l^2}{3\,l - 4\,x}$ (1), loads to the left of this produce positive shears at x, loads to the right produce negative shears at x.

If x is larger than x_1 then $S = \frac{P\,x_1}{l^2}(4\,x - 3\,l)$. As long as $x < \frac{3}{4}\,l$ the shear is negative; that is, a load to the left of a section produces negative shears. For $x = \frac{l}{4}$, Equation (1) gives $x_1 = \frac{l}{2}$; for $x = 0$, Equation (1) gives $x_1 = \frac{l}{3}$; that is, while the section, the shear of which is considered, moves from the end to the center of the half-span. The load which produces a shear, 0, at this section, moves from a point distant $\frac{l}{3}$ to a point distant $\frac{l}{2}$ from the left end. The shear in the right half-span at the distance x from the right end of the span produced by a load distant x_1 from the left end of the span is $S = -\frac{P\,x_1}{l}\left(1 - \frac{4\,x}{l}\right)$. This expression is negative as long as $x < \frac{l}{4}$, and is positive for $x > \frac{l}{4}$. That is, a load in one half-span produces negative shears at any point between the outside end and the center of the other half-span, and positive shears between the center of the other half-span and the center of the bridge.

Fig. 7 gives the shear curves due to a load, P, in different positions on the bridge. Figs. 6 and 7 will assist in determining the positions of the load giving the largest moments and shears.

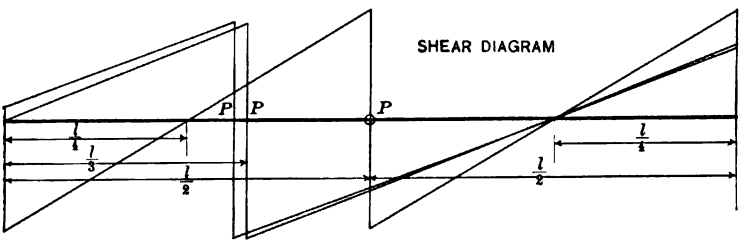

FIG. 7.

In the following derivation of the formulas for maximum moments and shears, a load of 6 000 lbs. per linear foot of truss, not more than 1 000 ft. long, will be assumed.

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I. Positive Moments Producing Compression in the Top Chords.—(See Fig. 8.)—To obtain the largest moment at the distance x from the

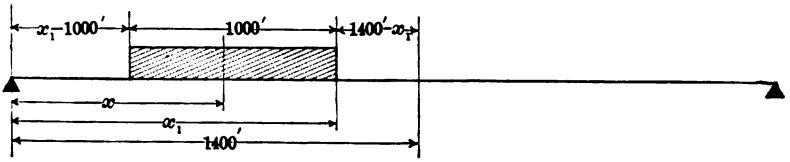

FIG. 8.

left end, the moving load must not cover those parts of the span where it would produce negative moments at x. It may not be long enough, however, to cover the whole of those parts of the span where it does produce positive moments at x. The load is assumed to extend from a point distant $x_1 - 1\,000$ ft. from the left end to a point distant x_1 from the left end, and the moment at the point distant x from the left end is expressed as a function of x_1 and differentiated in regard to x_1 to find the value of x_1, which gives the maximum moment at x. The equations of equilibrium are:

$$2\,800\,p + R - 1\,000\,w - R_1 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

$$p\,\frac{1\,400^2}{2} + R_1\,1\,400 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

$$p\,\frac{1\,400^2}{2} + R\,1\,400 - 1\,000\,w\,(1\,900 - x) = 0 \quad \ldots\ldots\ldots (10)$$

In which p = suspender pull per linear foot;
w = load per linear foot.

From these we obtain:

$$R_1 = \frac{5\,w}{14}\,(x_1 - 500) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

$$35\,p = \frac{w}{56}\,(x_1 - 500) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

$$R = \frac{w}{28}\,(43\,000 - 30\,x_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

$$M = R\,x + \frac{p\,x^2}{2} - (x - x_1 + 1\,000)^2\,\frac{w}{2}$$

$$M_x = w\,x\left[\frac{43\,000}{28} - \frac{15}{14}\,x_1\right]$$

$$+ \frac{w\,x^2}{3920}\,(x_1 - 500) - \frac{w}{2}\,(x - x_1 + 1\,000)^2 \quad \ldots\ldots\ldots (14)$$

$\frac{d\,M_x}{d\,x_1} = 0$ gives us $x_1 = 1\,000 + \frac{x}{3920}\,(x - 280)$.

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In the foregoing derivation, $x_1 - 1\,000$ was assumed as positive. That this may be so, x must be equal to or larger than 280 ft. If we let $x = 35\ \alpha$, where α is the panel index counting from the end, we obtain $\alpha \gtrless 8$. If we introduce $35\ \alpha$ instead of x in the foregoing equation for x_1 we obtain

$$x_1 = 1\,000 + \frac{5}{16}\ \alpha\ (\alpha - 8) \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

If we introduce this value in Equation (14) and replace x by $35\ \alpha$ and w by 3 tons we obtain:

$$M_x = \frac{15}{4}\ \alpha \left[13\,000 - \frac{150}{16}\ \alpha\ (\alpha - 8) \right]$$
$$+ \frac{15}{16}\ \alpha^2 \left[500 + \frac{5}{16}\ \alpha\ (\alpha - 8) \right]$$
$$- \frac{3}{2} \left[35\ \alpha - \frac{5}{16}\ \alpha\ (\alpha - 8) \right]^2$$

The third line is $= \dfrac{75\ \alpha^2}{512}\ (120 - \alpha)^2$

We obtain, by introducing this value and reducing (for $\alpha \gtrless 8$),

$$M\ \alpha = 75\ \alpha \left[650 + \frac{\alpha}{16} \left\{ 100 - \frac{(120 - \alpha)\ (104 + \alpha)}{32} \right\} \right] \ldots (16)$$

We obtain for R, R_1 and p

$$35\ p = \frac{3}{56} \left\{ 500 + \frac{5}{16}\ \alpha\ (\alpha - 8) \right\} = \frac{15}{896} \left\{ 1\,600 + \alpha\ (\alpha - 8) \right\} . (17)$$

$$R = \frac{3}{28} \left[13\,000 - \frac{150}{16}\ \alpha\ (\alpha - 8) \right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

$$R_1 = \frac{300}{896} \left[1\,600 + \alpha\ (\alpha - 8) \right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

For $\alpha < 8$ the load producing maximum positive moments extends from the end of the span to x_1. (See Fig. 9.)

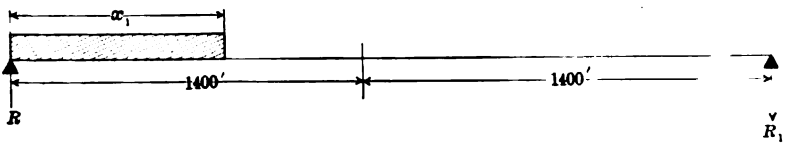

FIG. 9.

The three equations of equilibrium are:

$$2\,800\ p + R - R_1 - w\ x_1 = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

$$p\ \frac{1\,400^2}{2} - R_1\ 1\,400 = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$p\ \frac{1\,400^2}{2} + R\ 1\,400 - w\ x_1 \left(1\,400 - \frac{x_1}{2} \right) = 0 \ldots\ldots\ldots (22)$$

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From these we obtain

$$35\,p = \frac{w\,x_1^2}{80 \times 1\,400} \qquad (23)$$

$$R_1 = \frac{w\,x_1^2}{5\,600} \qquad (24)$$

$$R = w\,x_1\left(1 - \frac{3\,x_1}{5\,600}\right) \qquad (25)$$

$$M_x = R\,x + \frac{p\,x^2}{2} - \frac{w\,x^2}{2} = x\left[R + \frac{x}{2}\left(p - w\right)\right]$$

$$M_x = w\,x\left[x_1 + \frac{x_1^2}{2\,800^2}\left(x - 4\,200\right) - \frac{x}{2}\right] \qquad (26)$$

If we differentiate after x_1 and let $\frac{d\,M\,x}{d\,x_1} = 0$, we obtain

$$x_1 = \frac{40 \times 2\,800}{120 - \alpha} \qquad (27)$$

If we introduce this value of x_1 into Equation (26) we obtain, for $\alpha < 8$:

$$M\,\alpha = 105\,\alpha\left[\frac{40 \times 2\,800}{120 - \alpha} + \frac{1\,600 \times 35}{(120 - \alpha)^2}(\alpha - 120) - \frac{35\,\alpha}{2}\right],$$

$$M\,\alpha = \frac{105\,\alpha}{120 - \alpha}\left[20 \times 2\,800 - \frac{35\,\alpha}{2}(120 - \alpha)\right],$$

$$M\,\alpha = \frac{1\,837.5\,\alpha}{120 - \alpha}\left[3\,200 - 120\,\alpha + \alpha^2\right],$$

$$M\,\alpha = \frac{1\,837.5\,\alpha\,(80 - \alpha)\,(40 - \alpha)}{120 - \alpha} \qquad (28)$$

from (23) and (27) $$p = \frac{9\,600}{(120 - \alpha)^2} \qquad (29)$$

from (25) and (27) $$R = \frac{336\,000\,(60 - \alpha)}{(120 - \alpha)^2} \qquad (30)$$

from (24) and (27) $$R_1 = \frac{672\,000}{(120 - \alpha)^2} \qquad (31)$$

Negative Moments.—(See Fig. 10.)—The equations of equilibrium are

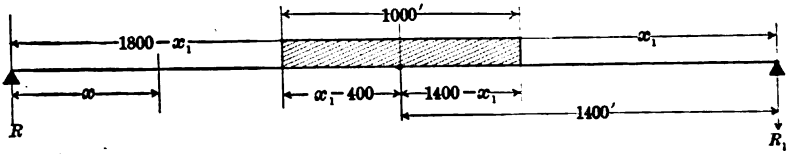

FIG. 10.

$$-R - R_1 + 2\,800\,P - 3\,000 = 0 \qquad (32)$$

$$-R\;1\,400 + \frac{p\,1\,400^2}{2} - \frac{3\,(x_1 - 400)^2}{2} = 0 \qquad (33)$$

$$-R_1\,1\,400 + p\,\frac{1\,400^2}{2} - \frac{3\,(1\,400 - x_1)^2}{2} = 0 \qquad (34)$$

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From these we obtain

$$R = \frac{3}{1\,400}\left\{ - x_1^2 + 1\,300\ x_1 + 90\,000 \right\} \quad \ldots\ldots\ldots\ldots (35)$$

$$R_1 = \frac{3}{1\,400}\left\{ - x_1^2 + 2\,300\ x_1 - 810\,000 \right\} \quad \ldots\ldots\ldots\ldots (36)$$

$$p = \frac{3}{1\,400^2}\left\{ - x_1^2 + 1\,800\ x_1 + 340\,000 \right\} \quad \ldots\ldots\ldots\ldots (37)$$

We obtain, therefore, for $M_x = - R\,x + \frac{p\,x^2}{2}$, substituting p and R from Equations (37) and (35),

$$M_x = - \frac{3\,x}{1\,400}\left\{ - x_1^2 + 1\,300\ x_1 + 90\,000 \right\}$$
$$+ \frac{3\,x^2}{2 \times 1\,400^2}\left\{ - x_1^2 + 1\,800\ x_1 + 340\,000 \right\} \quad \ldots\ldots\ldots (38)$$

Differentiating after x_1 and letting $\frac{d\,M_x}{d\,x_1} = 0$, we obtain, after introducing 35α for x, $x_1 = \frac{100\,(520 - 9\,\alpha)}{80 - \alpha}$ $\ldots\ldots\ldots\ldots\ldots\ldots (39)$

If we introduce this value of x_1 into Equation (38) we obtain,

$$M\alpha = - \frac{3\,\alpha}{40}\left\{ - \frac{10\,000\,(520 - 9\,\alpha)^2}{(80 - \alpha)^2} + \frac{130\,000\,(520 - 9\,\alpha)}{80 - \alpha} + 90\,000 \right\}$$
$$+ \frac{3\,\alpha^2}{2 \times 40^2}\left\{ - \frac{10\,000\,(520 - 9\,\alpha)^2}{80 - \alpha)^2} + \frac{180\,000\,(520 - 9\,\alpha)}{80 - \alpha} + 340\,000 \right\}$$

$$M\alpha = - \frac{75\,\alpha}{8}\left\{ 720 - 34\,\alpha + \frac{(520 - 9\,\alpha)^2}{80 - \alpha} \right\} \quad \ldots\ldots\ldots\ldots\ldots (40)$$

Introducing x_1 from Equation (39) into Equation (35) we obtain after reduction,

$$R = - \frac{150}{7}\left\{ 9 + \frac{4\,(520 - 9\,\alpha)\,(130 - \alpha)}{(80 - \alpha)^2} \right\} \quad \ldots\ldots\ldots\ldots\ldots (41)$$

Intrdoucing x_1 from Equation (39) into Equation (37) we obtain, after reduction,

$$p = \frac{3}{196}\left\{ 34 + \frac{(520 - 9\,\alpha)\,(920 - 9\,\alpha)}{80 - \alpha)^2} \right\} \quad \ldots\ldots\ldots\ldots\ldots (42)$$

Shears.—Positive shears $x < \frac{l}{4}$.

The load extends from x to the right to a point distant $x_1 = \frac{l^2}{3\,l - 4\,x}$ from the left end of the span. (See Fig. 11.)

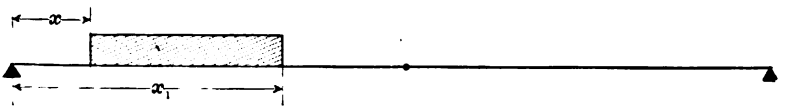

FIG. 11.

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The length of the load, $x_1 - x = \dfrac{l^2 - 3\,l\,x + 4\,x^2}{3\,l - 4\,x}$

The equations of equilibrium are:

$$R + p\,l - R_1 - w\,\frac{l^2 - 3\,l\,x + 4\,x^2}{3\,l - 4\,x} = 0 \quad \ldots\ldots(43)$$

$$R\,\frac{l}{2} + \frac{p\,l^2}{8} - w\,\frac{l^2 - 3\,l\,x + 4\,x^2}{3\,l - 4\,x} \times \frac{l - x_1 - x}{2} = 0 \quad \ldots\ldots(44)$$

$$R_1\,\frac{l}{2} - \frac{p\,l^2}{8} = 0 \quad \ldots\ldots(45)$$

From these we obtain, after introducing $x = 35\,\alpha$, $l = 2\,800$, $w = 3$;

$$R = \frac{21\,(1\,600 - 60\,\alpha + \alpha^2)\,(4\,800 - 340\,\alpha + 3\,\alpha^2)}{32\,(60 - \alpha)^2} \quad \ldots\ldots(46)$$

$$R_1 = \frac{21\,(1\,600 - 60\,\alpha + \alpha^2)\,(80 - \alpha)\,(20 + \alpha)}{32\,(60 - \alpha)^2} \quad \ldots\ldots(47)$$

$$p = \frac{3\,(1\,600 - 60\,\alpha + \alpha^2)\,(80 - \alpha)\,(20 + \alpha)}{3\,200\,(60 - \alpha)^2} \quad \ldots\ldots(48)$$

$$\alpha < 20;\ x < \frac{l}{4}.$$

Positive Shears.— $\alpha > 20$, $x > \dfrac{l}{4}$. (See Fig. 12.)

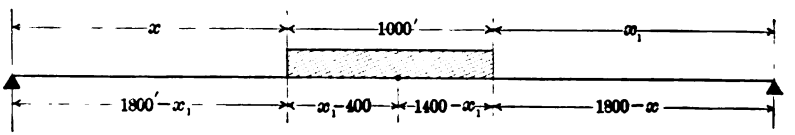

FIG. 12

For this position of load we found, under the head of negative moments,

$$R = \frac{3}{1\,400}\left\{-x_1^2 + 1\,300\,x_1 + 90\,000\right\}$$

$$p = \frac{3}{1\,400^2}\left\{-x_1^2 + 1\,800\,x_1 + 340\,000\right\}$$

We obtain from this, by introducing for x_1 its equivalent $1\,800 - x$, and letting $x = 35\,\alpha$, after reduction

$$R = \frac{3}{8}\,\alpha\,(460 - 7\,\alpha) - 1735.7$$

$$35\,p = \frac{3}{320}\,\alpha\,(360 - 7\,\alpha) + 18.214$$

$$\alpha > 20.$$

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Negative Shears.— $\alpha < 15$. (See Fig. 13.)

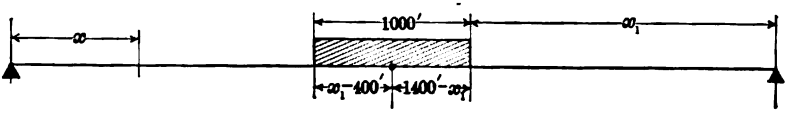

FIG. 13.

We have found previously, for this position of load,

$$R = \frac{3}{1\,400}\left\{ - x_1^2 + 1\,300\,x_1 + 90\,000 \right\} \quad \ldots\ldots(49)$$

$$p = \frac{3}{1\,400^2}\left\{ - x_1^2 + 1\,800\,x_1 + 340\,000 \right\} \quad \ldots\ldots(50)$$

$$S_x = - R + p\,x = \frac{3}{1\,400}\left\{ + x_1^2 - 1\,300\,x_1 - 90\,000 + \frac{x}{1\,400} \left(- x_1^2 + 1\,800\,x_1 + 340\,000 \right) \right\}$$

If we differentiate in regard to x_1, and let $\frac{d\,S\,x_1}{d\,x} = 0$, we obtain, after reduction, $x_1 = 100\,\frac{260 - 9\,\alpha}{40 - \alpha}$. Introducing this value of x_1 in Equations (49) and (50), we obtain

$$R = - \frac{750}{7}\,\frac{9\,\alpha^2 - 820\,\alpha + 16\,400}{(40 - \alpha)^2}$$

$$35\,p = \frac{75}{28}\,\frac{23\,\alpha^2 - 1\,840\,\alpha + 34\,800}{(40 - \alpha)^2}$$

Negative Shears: Second Alternative of Loading.—(See Fig. 14). $15 \leqq \alpha < 29$. For this loading we found previously:

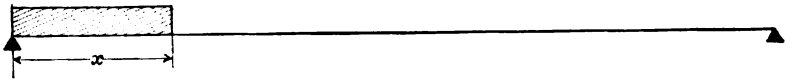

FIG. 14.

$$R = w\,x\left(1 - \frac{3\,x}{5\,600}\right) = 21\,\alpha\,\frac{160 - 3\,\alpha}{32}$$

$$35\,p = \frac{w\,x^2}{80 \times 1\,400} = \frac{21\,\alpha^2}{640}.$$

A numerical calculation shows that this second kind of loading for negative shears gives larger results than the loading above given for $\alpha \geqq 15$.

Negative Shears.—(See Fig. 15.) $x > 1\,000$, $\alpha \geqq 29$. The equations of equilibrium are

$$R + 2\,800\,p - 1\,000\,w - R_1 = 0 \quad \ldots\ldots(51)$$

$$R_1\,1\,400 - 1\,400 \times 700\,p = 0 \quad \ldots\ldots(52)$$

$$R\,1\,400 - 1\,000\,w\,(1\,900 - x) + p\,1\,400 \times 700 = 0 \quad \ldots\ldots(53)$$

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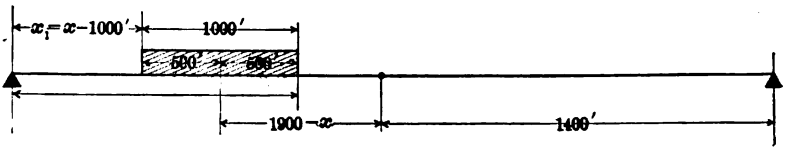

FIG. 15.

From these we obtain $p = \frac{3\,(7\,\alpha - 100)}{392}$,

$$35\,p = \frac{15}{16}\,(7\,\alpha - 100), \qquad R = \frac{75}{14}\,[860 - 21\,\alpha].$$

For obtaining the maximum strains in the diagonals, several values of α have sometimes to be tried.

ON THE DIMINUTION OF THE MOVING-LOAD STRESSES IN STIFFENING TRUSSES BY THEIR DEFORMATION.

The stiffening trusses will deflect under the influence of the moving load; in consequence, the cables, which are parabolas as long as the stiffening trusses are straight, will change their shape, and the loads per linear foot of bridge carried by them will not be uniform over the whole length of the span. Where the stiffening trusses deflect downward, the load on the cables will be more, and where they deflect upward it will be less, than it would be if the stiffening trusses did not change their form.

For the purpose of obtaining as good an approximation to the actual stresses as is easily practicable, it will be assumed that the loads on the cables are uniform above the moving load 1 000 ft. long, and that they are also uniform above the unloaded part of the bridge. The vertical deformation of the cables will be very nearly the same as the vertical deformation of the stiffening trusses below them, as the slight change in length of the suspenders, due to the unequal stresses in them, above the loaded and unloaded part of the bridge may be neglected. To obtain the difference in load on the cables above the loaded and above the unloaded parts of the bridge, we equate the deflection produced in the cables by the difference in the loads on different parts of them, and the deflection of the stiffening trusses produced by their stresses. For the sake of simplicity we determine the deflection of both the cables and the stiffening trusses 700 ft. from the end of the loaded half span of the stiffening trusses, taking the span of the cables to be 2 800 ft. and the deflection one-eighth of the span. We further assume the

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moving load 1 000 ft. long to be in the center of the half span. All the dead load, being uniform over the whole span, is carried by the cables, and produces no stress in the stiffening trusses.

In Fig. 16, the dotted curve gives the position of the cables under unequal loading, the full curve that under equal loading. The shaded area gives the load carried by the cables.

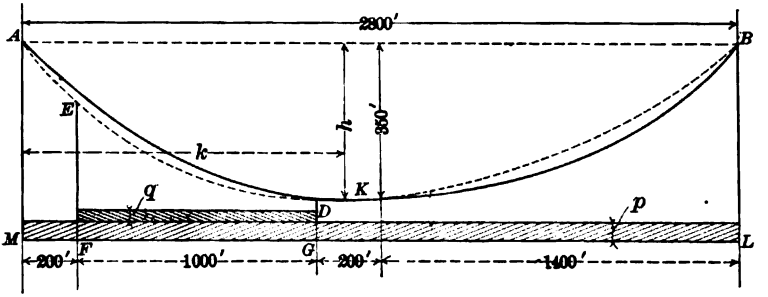

FIG. 16.

If the moving load per linear foot is r, extending from F to G, then we have, in accordance with the above assumptions a uniform load, p, per linear foot of bridge acting on the parts $A\,E$ and $D\,B$ of the cables and a larger load, say, $p + q$ per linear foot of bridge, acting on the part $E\,D$ of the cables. On the stiffening trusses we have, on the parts $M\,F$ and $G\,L$, one uniform upward pull per linear foot of bridge, and another uniform downward pull per linear foot of bridge over the part $F\,G$. On this latter part, the upward force from the cables per linear foot of bridge is larger by the amount, q, than on the remainder of the span; the downward force from the loads is larger by the amount, r, per linear foot of bridge than on the remainder of the span. The resultant downward force on $F\,G$ is different from the force acting on the remainder of the stiffening truss by the amount, $r - q$, per linear foot of bridge. If the load per linear foot were the same over the whole length of the cables, then this difference would be r instead of $r - q$. That is, the effect of the difference, q, in the load per linear foot on the different parts of the cables is equivalent to an equal diminution in the moving load acting on the stiffening trusses.

For the purpose of obtaining the change in the deflection of the cables:

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Let R = the vertical reaction at the top of the left tower, due to the cables of the main span;

H = the horizontal reaction;

K = the lowest point of the cables;

h = the dip of the cables;

k = the distance of the lowest point of the cables from the left tower.

Then $R = 1\,400\ p + 750\ q$; also, $R = p\ k + 1\,000\ q$; and from these two equations we obtain:

$$k = 1\,400 - 250\,\frac{q}{p};$$

$$\text{also } H h_1 = R\ k - \frac{p}{2}\,k^2 - 1\,000\ q\ (k - 700).$$

After introducing the values for R and k from the foregoing equations, we obtain $H = \frac{p}{2\,h}\left(1\,400 + 250\,\frac{q}{p}\right)^2$

The equation of the parabola $E\,D$, with A as origin, $A\,B$ as axis of x, and $A\,M$ as axis of y, is:

$$R\,x - H\,y - \frac{p\,x^2}{2} - \frac{q}{2}(x - 200)^2 = 0.$$

After introducing R and H from the foregoing equations, we obtain for the ordinate y_1 of the point of the parabola, $E\,D$, distant 700 ft. from the left tower:

$$y_1 = h\,\frac{588 + 320\frac{q}{p}}{\left(28 + 5\,\frac{q}{p}\right)^2}.$$

If the same amount of load on the cables as above assumed were equally distributed over their length, the dip would be h_1, and the ordinate of the points of the cables 700 ft. from the ends of the span would be $y_2 = \frac{3}{4}\,h_1$.

If h_1 is calculated it is found to be very nearly equal to h, so that the difference, $h - h_1$, may be neglected. The change in the ordinate of the cables 700 ft. from the left tower, therefore, is

$$y_1 - y_2 = h\,\frac{588 - 320\,\frac{q}{p}}{\left(28 + 5\,\frac{b}{p}\right)^2} - \frac{3}{4}\,h.$$

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If we let $y_1 - y_2 = \delta$, and $\frac{q}{p} = t$, and find t from the foregoing equation, we obtain

$$t = \frac{55\,h - 140\,\delta - 55\,\sqrt{h\,(h - 9.95\,\delta)}}{18.75\,h + 25\,\delta}$$

δ is the change in the ordinate of the cables 700 ft. from the left tower, due to their unequal loading as compared with equal loading of the same amount. This must be equal to the deflection of the stiffening trusses directly below, caused by the stresses due to the given loading.

This deflection, calculated from a stress sheet of the stiffening trusses, is 2.5 ft., and h is equal to 350 ft. If these values are introduced into the foregoing formula for t, we obtain:

$$t = \frac{q}{p} = 0.05232; \quad \text{or } q = 0.05232\,p.$$

The dead load per linear foot of bridge on each side of the axis is 14 000 lbs., and the corresponding moving load is 6 800 lbs. per linear foot of bridge. The latter is 1 000 ft. long. Since the downward forces acting on each truss are equal to the upward forces acting on it, we have:

$$R + R_1 + 6\,800 \times 1\,000 + 14\,000 \times 2\,800 = q \times 1\,000 + p\,2\,800.$$

The two end reactions R and R_1 of the stiffening trusses are alike and opposite for this kind of loading. After the introduction of the value of q from the previous equation and reduction, the equation, therefore, becomes $p = 16\,128$ lbs.; and, from this and the previous equation, $q = 844$ lbs. The result, therefore, is that the cables on each side of the axis distribute 844 lbs., and each stiffening truss 5 956 lbs. of the total moving load of 6 800 lbs. per linear foot of bridge.

As can be shown by a lengthy argument, which the writer will not introduce here, the assumption made, that the loads acting on the cables are uniform over the loaded part of the span, and also uniform but different over its unloaded part, is not quite correct. The method here used for finding the part of the moving load which is distributed by the cables, therefore, is approximate only.

In the case of deep stiffening trusses, the diminution of the stresses from moving load, in consequence of the deflection of these trusses, amounts to about one-eighth of the total stress which would exist if the trusses were rigid, or to one-seventh of the stresses obtained in the stiffening trusses. An error of 10% in this correction, therefore,

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amounts to only 1.4% of the stress obtained. The method, therefore, may be used for deep trusses without serious error. For very shallow trusses it would be objectionable, as the error there would be a large fraction of the final result. Very shallow trusses, however, can only be used for highway bridges. Center hinges in them are superfluous.

The results here obtained, in regard to the relative economy of three-hinged and two-hinged stiffening trusses and braced chains, apply only to bridges of about the dimensions of the example chosen for the comparison.

The formulas given for the calculation of the moving-load stresses in a stiffening truss of three hinges apply only to a truss of 2 800 ft. span with 80 panels of 35 ft., and with a moving load of 3 tons per foot, 1 000 ft. long or less. The formulas can easily be changed to suit a different number of panels or a different amount of load. The manner of obtaining the formulas is given, and can be applied to any length of span and any length of load.

The formulas given have been repeatedly checked, first by Mr. E. Ludlow Gould, then by Mr. Edwin Duryea, Jr., and by Mr. Alfred Noble. They were used in making the stress sheets repeatedly referred to.

The Proper Unit Stresses in the Stiffening Trusses.

The trusses here considered differ from ordinary trusses, for which the usual unit stresses are appropriate, not only by the unprecedented span, but by other characteristics which have an important bearing on the selection of the unit stresses. All members of these trusses are subject to opposite stresses of nearly equal amounts. The stresses of one kind are the consequence of upward pulls by the suspenders above the unloaded parts, which are in excess of the weights directly below. These stresses reach the trusses in a very indirect way, as they travel from the moving loads by the nearest suspenders into the cables, through these to other suspenders and through them to the unloaded parts of the trusses. The maximum stress of one kind, even in the floor-beam suspenders, is due to the co-operation of loads on at least six tracks. The maximum stress of the other kind is due to a definite position of ten trains of maximum weight. In all other members, except the floor-beam suspenders (which need special unit

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stresses), both kinds of stresses only occur by the coincidence of ten trains of maximum weight in a definite position on the bridge. A peculiarity of the stiffening trusses is that any load in the wrong place diminishes the stresses instead of increasing them.

To assist in the judicious selection of the unit stresses the writer will give in figures a rough approximation to the frequency of occurrence of given fractions of the maximum stresses. For this purpose an estimate of the number of trains of maximum weight is required.

The freight trains of 1 500 tons would, if ever, rarely go to the city, and would probably never leave the city. It will be assumed, however, that two such trains will pass each way daily, and 150 trains per day of half this weight on each track.

On the long-distance passenger tracks, it will be assumed that there are on each track 20 daily trains of 750 tons, and 120 daily trains of half the weight.

On the six tracks connected with the elevated and underground roads in New York it will be assumed that there are on each track 60 trains of maximum weight per hour for four hours each day, and 20 trains of half weight per hour for the remainder of the day.

These assumptions are somewhat arbitrary, but, undoubtedly err by excess rather than deficiency. The street-car tracks will have practically uniform loads over the whole span, and need not be considered here.

Each train will produce maximum stresses in any one member only in one position; but it will produce stresses near the maximum in neighboring positions. All trains will produce important stresses of one kind in any given member during about 800 ft. of their motion. A speed of 40 ft. per second will be assumed. Each train, therefore, will produce important stresses during not more than 20 seconds. All the freight trains are assumed to be crowded into 12 hours each day. The two freight tracks are assumed to be in the center between the trusses; the two long-distance passenger tracks are assumed to be on one side, between the freight tracks and the trusses.

There would be one eastward freight train of maximum weight in a position to produce its share of the maximum stress of one kind in any member for less than 40 seconds during each 12 hours, or for $\frac{1}{1080}$ of the total time. There would be two freight trains of maximum

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weight in the proper position on the bridge to produce their share of the maximum stress of one kind in any member of the trusses for $\frac{1}{1080^2}$ of the total time. While the former event occurs twice a day, the latter occurs only once in 540 days, or once in 1.48 years. This combination produces less than 44.1% of the maximum stress in the stiffening trusses.

On the long-distance passenger track near one truss there is one train of maximum weight for 400 seconds every 24 hours, or for $\frac{1}{216}$ of the total time. This train produces a stress in the rear truss of less than 18.22% of the maximum stress during this time. It coincides in the proper position with the two freight trains producing 44.1% of the maximum stress less than once in $1.48 \times 216 = 320$ years. The stress due to this coincidence is less than 62% of the maximum stress.

A light passenger train on the other long-distance passenger track is for $\frac{1}{36}$ of the time in the proper position on the bridge to produce a stress not exceeding 9.8% of the maximum stress. A stress of 72% due to this combination occurs less frequently than once in $320 \times 36 = 11\ 520$ years. A heavy suburban train on the track nearest the truss considered occurs during four hours each day for one-third of the time in a position to produce a stress not exceeding 5.5% of the maximum stress. A stress of 77.5% due to the combination of all the preceding trains occurs less than once in $11\ 520 \times 9 = 103\ 680$ years.

If we take heavy freight trains on the two freight tracks and light passenger trains on the passenger tracks near the truss considered, we find that 56% of the maximum stress occurs less than once in 27 years.

If we take the four hours of heavy suburban traffic, we find that there is one train in proper position to produce large stresses for $\frac{1}{3}$ of the time, two trains for $\frac{1}{9}$ of the time, 3 trains for $\frac{1}{27}$ of the time, 4 trains for $\frac{1}{81}$ of the time, 5 trains during $\frac{1}{243}$ of the time, and 6 trains during $\frac{1}{729}$ of the time. The last combination occurs once a day, and it produces 45% of the maximum stress in one of the trusses. The two heavy freight trains coincide during these four hours once in $4\frac{1}{2}$ years. A stress equal to 89% of the maximum occurs less than once in $4.5 \times 729 = 3\ 180$ years. If the heavy passenger train on the track nearer this truss is combined with the last combination we obtain 96% of the maximum stress once in $3\ 180 \times 216 = 687\ 000$ years. The maximum

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stress itself, according to the assumptions made, would occur once in 687 000 × 216 = 148 350 000 years. The frequency of occurrence of the various percentages of the maximum stress here given, is evidently much exaggerated, as it has been assumed that each train will produce its greatest effect for 20 seconds.

The experiments, which show that by the frequent application of opposite stresses it is possible to break a bar with a much smaller stress than if only one kind of stress is used, therefore, have no bearing on this case. The theoretical maximum stress may occur once in the lifetime of the bridge, but even this is extremely improbable. One is therefore amply justified in dimensioning the stiffening trusses by considering only that maximum stress which requires the larger section. Impact in the main diagonals and chords of this bridge is a practically negligible quantity. The maximum wind stresses are also of rare occurrence, and their coincidence with the maximum moving-load stresses is extremely remote.

The maximum stresses, therefore, may be treated as if they were dead-load stresses of only one kind. For dead-load stresses two-thirds of the elastic limit of test specimens is generally considered a safe stress. The steel in the stiffening trusses has been assumed to have an ultimate strength of 80 000 to 90 000 lbs., and an elastic limit of 45 000 lbs. per square inch. A stress of 30 000 lbs., therefore, would be a safe unit stress, if all the stresses were considered. The weight of the members, and the wind pressure on them, produce bending stresses which were not considered in dimensioning. To allow for these a stress of 27 000 lbs. per square inch, net section, has been used for tension, and $27\,000 - 108\,\frac{l}{r}$ was taken as the unit stress in compression. For moving load alone the corresponding unit stresses were 22 500 and $22\,500 - 90\,\frac{l}{r}$.

For the cables, wires of an ultimate strength of at least 200 000 lbs. and an elastic limit of 170 000 lbs. were assumed. The wire couplings should have at least 95% of the strength of the wires.

If the cables are properly designed, so that the bending stresses in them are of small amount and known with certainty, a unit stress of 70 000 lbs. per square inch is perfectly safe, if the bending stresses are neglected.

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DISCUSSION.

Mr. Moisseiff. L. S. MOISSEIFF, Assoc. M. Am. Soc. C. E.—The literature on the subject of suspension bridges which is available for the design of long-span bridges of this type is very meager, especially in the English language. The information given in standard text-books on bridges is either obsolete, in its cultivation of old theories, or, where more modern methods are followed, this information is so scant as to prove of little practical value. The engineer is compelled to review old reports and reprints, many of which contain errors, typographical and original. But, more serious is the notable absence of any numerical data on suspension bridges of spans exceeding 1 000 ft.

A suspension bridge, in its action, is such that the effect of moving load, temperature, etc., is generally not represented by linear equations. This means that the stresses do not vary in simple proportion to the applied forces. Hence, approximations and professional guesses are not so easily made. The paper presented by Mr. Mayer, therefore, will be much appreciated by the members of this Society. While chiefly discussing the question of three-hinged *versus* two-hinged stiffening trusses, it gives numerical values and averages for the longest span suspension bridge yet contemplated. Therefore the absence of a systematical tabulation and the lack of explanatory diagrams will be much regretted. The addition of a few sketches and some additional information would greatly relieve the paper of the mysterious air now hovering over it. For instance, the cross-section of the cable is nowhere given, and, while it is not a matter of much difficulty to determine the same, such information should certainly be given. As presented, in order to make some simple comparisons, the reader has to make a diligent search for the data.

While giving the numerical results of a concrete case, the author, in many instances, has failed to give the intermediate steps in their derivation, and, mainly, the assumptions on which they are based. Just comparisons of these results with results otherwise obtained are thereby made impossible. This applies especially to the values given to the deflections under different conditions. The notation used is also unsystematic. American engineers are used to struggling with much wilful notation, and are accustomed to finding in each book different symbols denoting the same thing; still, they will find the author's notation of horizontal wind pressures per linear foot, as x and y, somewhat out of the usual.

In finding the "Stresses Produced by the Change in Length of the Cables in Suspended Stiffening Trusses of Three Hinges," the author uses the formulas deduced by him in *Engineering News*. These formulas are approximate only, as stated by himself.

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Mr. Moisseiff.

It used to be an axiom, in the treatment of three-hinged arches and suspension bridges, that stresses due to changes of temperature, elongation of cables and displacement of the points of support, did not affect the stresses in these structures. Their deformations were considered to be insignificant, and their apparent statical determinateness eliminated all elastic equations. This was true within certain limits, namely, when the spans were short, and the trusses stiff enough to make the deformations, due to the above causes, so small as to be negligible, thus approaching the statical assumption of rigidity. But the long-span suspension bridge, with its comparatively flexible stiffening truss, does not generally come within these limits. Stresses due to elongations of cables will be caused in three-hinged as well as in two-hinged trusses.

In the report of the "Board of Engineer Officers as to Maximum Span Practicable for Suspension Bridges," G. Lindenthal, M. Am. Soc. C. E., gives the first analytic treatment of the subject of "Temperature Strains in Three-Hinged Arches." After stating that the old theory of freedom from temperature stresses is erroneous, he continues:

"There are, of course, no temperature strains in the middle hinge, but they do exist for any change from the normal temperature in the arches between the end and middle hinges, and are too large to be neglected in the computations."

This, of course, would also be true for any elongation of the cables. In the succeeding proof of the foregoing statement, Mr. Lindenthal assumes that the new curve of the deflected bridge will also be parabolic, and it follows that "the maximum bending moment at the middle half-girder is as large as in the middle of the continuous girders."

Professor Joseph Melan, the well-known authority on suspension bridges, has also taken up the subject.* After stating that a rigid treatment of the problem may be made, but is difficult and cumbersome, Professor Melan approaches the subject by also assuming the new line of equilibrium to be a continuous parabola. Assuming the neutral axis to bend in a parabolic curve, it follows that the maximum temperature stress at the center of the half-span of the three-hinged bridge is 80% of the maximum bending moment of the end-hinged stiffening truss. Professor Melan states that these stresses are actually excessive, "as the cable does not remain parabolic, but, due to the unequal distribution of suspender stresses, it forms a kink at the center."

Mr. Mayer, in his analysis in *Engineering News*, on which the computation of temperature stresses is based, starts with another assumption: He assumes the trusses to remain rigid and to turn around

* *Zeitschrift des Oesterr. Ing. u. Arch. Vereins*, No. 36, 1900.

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Mr. Moisseiff. their end hinges. With the assumption of rigid trusses it is also tacitly assumed that the distribution of the load on the suspenders remains as before. The cable curves, therefore, will remain parabolic in their two halves, and will have a kink at the center. On this assumption the author's formula is based.

It will be noticed that the two different assumptions made in the analysis, and which lead to entirely different conclusions as to the importance of elongation stresses in three-hinged stiffening trusses, form the two extreme conditions of the case. The assumption of the deflected cable remaining one continuous curve will necessarily give higher values, as Professor Melan anticipates. The other assumption of the rigid truss will give values which are too low. The actual case will lie between these extreme assumptions. The truss, of course, will not be rigid, but will deflect and thereby change the curve of the cables from the presupposed parabolic form. The new curve of the cables will be a resultant curve intermediate between the parabola of the original curve and the neutral line of the deflected trusses. The new curve, in general, will consist of two catenarian halves, the tangents of which, at the center of the span, form an oblique angle; but, as the truss increases in stiffness, then more nearly will the new curve approach the two parabolic halves having a kink at the center.

In the following the writer will endeavor to establish the general equation of this curve. The general reasoning followed is analogous to that used by Professor Melan, in determining the true curve of continuous end-hinged stiffening trusses, in his work on "Suspension Bridges and Arches;" its application to the three-hinged truss is original with the writer.

The bridge is supposed to be free from all temperature and elongation stresses at a normal temperature and under the fixed load only. It is also supposed that then the curve of the cable is parabolic. Both assumptions are actually verified in practice.

Let H_f = the horizontal pull in the cables at the normal state under fixed load only;

S_o = the stress in the suspenders, per linear foot;

K = the weight of the cable, per horizontal foot.

Then the differential equation of the equilibrium polygon will be

$$H_f \frac{d^2 y}{d x^2} = -(S_o + K) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

When an additional load, q, per linear foot, comes on the bridge, or any other change takes place, the suspender stress changes to S_1, the horizontal pull to $(H_f + H)$, and the ordinate of the equilibrium polygon to $y + \eta$. These new quantities are again subject to the equation of condition.

$$-(H_f + H) \frac{d^2 (y + \eta)}{d x^2} = S_1 + K \ldots\ldots\ldots\ldots\ldots (2)$$

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η represents the deflection of the cable at any point x (Fig. 17). Meantime, the half-trusses have also been deflected. If we neglect the very small elongation of the suspenders due to the change, the deflection of the cable, $\eta = \delta + \frac{2}{l} \Delta f$. δ represents here the deflection of the truss due to bending, and Δf the deflection of the cables at the center hinge. The expression, $\frac{2x}{l} \Delta f$, represents the deflection of the truss due to its turning on its end hinge, as can be seen from the diagram. Mr. Moisseiff.

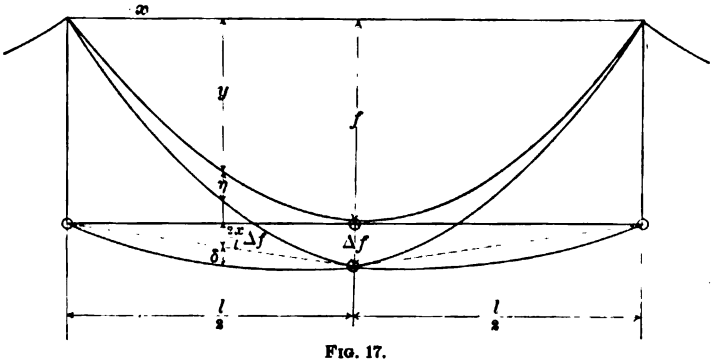

FIG. 17.

According to the common theory of flexure, $-EI \frac{d^2\delta}{dx^2} = M$, where I = moment of inertia, E = coefficient of elasticity, and M the bending moment. Since the load per unit of length is the second differential quotient of the bending moment,

$$EI \frac{d^4\delta}{dx^4} = q - [s_1 - s_0] \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The addition of the foregoing three equations gives:

$$EI \frac{d^4\delta}{dx^4} - (H_f + H) \frac{d^2\delta}{dx^2} - H \frac{d^2y}{dx^2} = q \ldots\ldots (4)$$

Writing for brevity, $\frac{H_f + H}{EI} = c^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$

and assuming I and q to be constant, within the limits of the integration, the differential equation (4) is integrated by the aid of a particular solution,

$$\frac{d^2\delta}{dx^2} = A e^{cx} + B e^{-cx} - \frac{H}{H_f + H} \left(\frac{q}{H} + \frac{d^2y}{dx^2} + \frac{1}{c^2} \frac{d^4y}{dx^4} + \ldots\ldots \right)$$

due to the original parabolic form $\frac{d^2y}{dx^2} = -\frac{8f}{l^2}$ and $\frac{d^4y}{dx^4} = 0$,

$$\text{then } \frac{d^2\delta}{dx^2} = A e^{cx} + B e^{-cx} - \frac{H}{H_f + H} \left(\frac{q}{H} - \frac{8f}{l^2} \right) \ldots\ldots (6)$$

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Mr. Moisseiff. The forces, $q - (s_1 - s_o)$, acting on the trusses through the suspenders cause a moment

$$M = M_o - \left(H_f + H\right)\left(\delta + \frac{2x}{l}\Delta f\right) - Hy \ldots\ldots (7)$$

where M_o represents the moment which the same load would produce in a simple supported beam of the same span, l.

$$\frac{d^2\delta}{dx^2} = -\frac{M}{EI} = c^2\eta - \frac{c^2}{H_f + H}(M_o - Hy) \ldots\ldots (8)$$

The comparison of equalities in Equations 6 and 8, and the introduction of $y = \frac{4f}{l^2}x(l-x)$, results in

$$\delta + \frac{2x}{l}\Delta f = \frac{H}{H_f + H}\times$$

$$\left[C_1 e^{cx} + C_2 e^{-cx} + \frac{8f}{c^2 l^2} - \frac{4f}{l^2}x(l-x) + \frac{M_o}{H} - \frac{q}{Hc^2}\right] \ldots\ldots (9)$$

C_1 and C_2 are constants, being, respectively, $\frac{AEI}{H}$ and $\frac{BEI}{H}$.

The values of C_1 and C_2 are determined in the usual way.

For $x = 0$, $\delta = 0$; for $x = \frac{l}{2}$, $\delta = 0$. This gives two new equations,

$$C_1 + C_2 + \frac{8f}{c^2 l^2} - \frac{q}{Hc^2} = 0,$$

$$C_1 e^{\frac{cl}{2}} + C_2 e^{-\frac{cl}{2}} + \frac{8f}{c^2 l^2} - \frac{q}{Hc^2} + \frac{q}{8H}l^2 - f - \Delta f\frac{H_f + H}{H} = 0,$$

writing

$$F = \frac{q}{8H}l^2 - f - \Delta f\frac{H_f + H}{H}.$$

Wherefrom, we obtain,

$$C_1 = \left[\frac{H + H_f}{H} + \frac{F}{1 - e^{-\frac{cl}{2}}} - \frac{8f}{c^2 l^2} + \frac{q}{Hc^2}\right]\frac{1 - e^{-\frac{cl}{2}}}{e^{\frac{cl}{2}} - e^{-\frac{cl}{2}}} \ldots\ldots (10)$$

$$C_2 = -C_1 - \frac{8f}{c^2 l^2} + \frac{q}{Hc^2} \ldots\ldots (11)$$

The greatest deflection of each half-span will be found at the quarter-point, $x = \frac{l}{4}$,

$$\text{Max. } \delta = \frac{H}{H_f + H}\times$$

$$\left[C_1 e^{\frac{cl}{4}} + C_2 e^{-\frac{cl}{4}} + \frac{8f}{C^2 l^2} - \frac{q}{Hc^2} + \frac{3}{4}\left(\frac{q}{8H}l^2 - f\right)\right] - \frac{\Delta f}{2} \ldots\ldots (12)$$

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Mr. Moisseiff.

The catenary character of the deflection curve is seen at once from the equations.

In the case of a span of 1 433 ft., a versed sine of 156 ft., stiffening trusses 35 ft., center to center of chords, and ratio of moving load to fixed load = 1 to 3.5, the numerical evaluation of the foregoing formulas showed a deflection at the middle of the half-span trusses equal to 10% of the deflection at the center hinge. The greatest moment in the center-hinged stiffening truss, due to the elongation of the cables, proved to be about 40% of the greatest moment of the end-hinged truss. As the equations show, these moments are also depending on the ratio of moving to fixed load in the bridge and on the stiffness of the trusses. As it might have been expected, the true equilibrium curve of the cables will lie between the two extreme assumptions made in the approximate analysis; so are also the stresses induced by the deflection between the two extreme values.

The author finds a deflection of his bridge at the center, under full load and maximum change of temperature, of ± 6.9 ft. He then determines the stress which would be caused by this deflection in continuous end-hinged stiffening trusses of 60 ft. and 140 ft. depth, respectively. This comparison is erroneous. A continuous truss, under the same conditions, will deflect less than one with a center hinge. While the center-hinged truss will not carry much load to the abutments as a truss, the one without a center hinge will, in some cases, carry considerable load. The cables will be relieved by the amount carried by the trusses, and, as a result, their deflection will be less.

Using the author's notation and assumptions, the load carried by the end-hinged truss is given by the following:

The greatest moment, due to the load, p, per linear foot, carried by the trusses, is $M = \frac{p\,l^2}{8}$. The constant stress per square inch in the truss assumed by the author will then be $t = \frac{M}{d\,A} = \frac{p\,l^2}{8\,d\,A}$, where $A =$ the area of each chord.

The author deduces: $t = \frac{4\,e\,\delta\,d}{l^2} = \frac{p\,l^2}{8\,d\,A}$; hence the load per linear foot carried by the truss, $= p = \frac{32\,d^2\,e\,A}{l^4}\,\delta$.

For a depth of truss, $d = 60$ ft., and an average area, as given by the author, $A = 727$ sq. ins.; the load carried by trusses results in $p =$ 565 lbs. per linear foot. This represents 6.7% of the total moving load.

For a depth, $d = 140$ ft., and the same section, $p = 3\,080$ lbs., or 36% of the moving load.

The cables will be relieved by the above amount, and their elongation due to moving load will be proportionately less. For the 140-ft.

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Mr. Moisseiff. truss, the deflection will be reduced by 1.15 ft., or about 17% of the total deflection assumed.

It would also appear that the pointed shape which the cables assume after the deflection of the center-hinged truss should, by its mere geometric shape, everything else being equal, give a deflection in excess of the deflection of a curve continuous over the center. It would not be surprising should it be found that the increased deflection of the three-hinged truss is quite a considerable percentage of the deflection of the continuous curve.

The whole argument of the paper is based on the deduced deflections and stresses due to the elongation of the cables by moving load and temperature. With a change in their values, the whole comparison of the relative economy in material is thrown out. It would appear, therefore, that computations nearer to the actual conditions should be made before conclusions as to the relative economy of three-hinged and two-hinged trusses are drawn.

The author states that "three-hinged, deep, stiffening trusses have the advantage that the maximum grade is only one-half of that which occurs if there are fewer than three hinges." This would appear to be of determining importance. If a three-hinged truss should really have one-half the maximum grade of the end-hinged truss, this fact would decide the case, for all long-span railroad bridges, against the two-hinged truss. Fortunately, such is not the case.

For the sake of general comparison, the grade will be expressed as an algebraic formula. It will also be assumed that the deflection is the same in all cases, which, as shown, is not true. For a three-hinged truss with rigid stiffening trusses, the grade is constant up to the center, and is given by tan. $\alpha = \frac{2\eta}{l}$ Fig. 18.

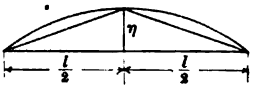

FIG. 18.

For a continuous truss, hinged at the ends, the following cases may be considered: If the bending of the truss is assumed to be parabolic, the maximum grade at the towers is tan. $\alpha = \frac{4\eta}{l}$. The grade, following the law of the parabola, will decrease to zero at the center. If the neutral curve is assumed to be circular, the same is a very close approximation. This is the author's assumption. It means that the truss, under full temperature and moving load, is a beam of constant stress at any point of its span.

For a beam of constant moment of inertia, the bending will follow the common neutral curve. According to the common theory of flexure, $M = E\,I\,\frac{d^2\eta}{dx^2}$, hence, tan. $\alpha_x = \frac{d\eta}{dx} = \int \frac{M}{EI}\,dx$, integrated between the limits of x, which makes tan. $\alpha_x = 0$ and tan. $\alpha_x =$ tan. α_x. For a

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Mr. Moisseiff.

full uniform load, $M = \frac{p}{2} x (l - x)$. This gives

$$\text{tan. } \alpha_x = \frac{d\eta}{dx} = \frac{p}{2EI}\left[\frac{lx^2}{2} - \frac{x^3}{3}\right]_0^{\frac{l}{2}} = \frac{p\,l^3}{24\,E\,I}.$$

The deflection of the same beam at the center under the uniform load, p, is, as is well known, $\eta = \frac{5\,p\,l^4}{384\,E\,I} = \frac{5\,l}{16} \times \frac{p\,l^3}{24\,E\,I} = \frac{5\,l}{16}$ tan. α.

Hence, tan. $\alpha = 3.2\,\frac{\eta}{l}$.

Now, the distribution of greatest moments due to combined moving load and elongation, in well proportioned two-hinged stiffening trusses, is such that the section of the chords is almost constant for the middle half. The conditions causing the maximum grade are due to uniform, or almost uniform, distribution of load on the truss. The sections of the chords are determined, not by this loading, but by partial moving load. The truss, therefore, evidently, will never be in the condition of constant stress assumed by the author. It will be much nearer to the assumption of a constant average moment of inertia.

Actual computations on the deflection of a two-hinged truss, of 1 433 ft. span and 35 ft. depth, show that a correction of about 6% of the deflection would bring the result to the same as for an average constant moment of inertia throughout. The grade at the towers will then be tan. $\alpha = \frac{3\,\eta}{l}$.

As was found from the true deflection curve deduced previously, well proportioned stiffening trusses of the three-hinged type will deflect at the quarter-points about 10% of the deflection at the center, or $\frac{\eta}{10}$. Supposing the assumption of circular bending to be true for center-hinged trusses, the greatest additional grade due to this deflection becomes $\frac{0.8\,\eta}{l}$. The total maximum grade of the three-hinged truss thus becomes $\frac{2.8\,\eta}{l}$, as compared to $\frac{3\,\eta}{l}$ of the two-hinged truss. The difference in grade between the two systems, then, is not 100%, but about 7%.

If the corrections of deflection discussed in the previous part of this discussion are made, the above small difference in grade will completely vanish, and it will probably appear that a three-hinged will have a steeper grade than a two-hinged truss. Really, it should not have needed so many figures to arrive at this conclusion.

Of course, all this discussion of the maximum grade is somewhat academical. The condition of extreme cold and no load on a span of such length will seldom, if ever, be realized. But the writer believes

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Mr. Moisseiff. that such a sweeping statement as to grade as that made by the author should be contradicted, as it conveys an erroneous notion.

The obvious result of the introduction of a center hinge is the throwing of more of the moving load on the cables. The same is true for the wind system. In a comparison of the relative economy of the two systems discussed, the effect of this addition on the cables should be considered. The writer has failed to see in the paper any mention of the additional load on the cables, and its effect, if any, on their section and increased weight.

The effect of the backstays in increasing the deflection of the main span shows some interesting features. Based on the figures given by the author, it will be easily found that, at mean temperature and with a fixed load, only the versed sine of the backstay of the long span of 1 740 ft. will be 29.8 ft. With the application of a moving load covering the whole span, the above backstay will have an upward deflection of 8.5 ft. This, of course, has a great influence in making up the deflection of the main span. The question then arises as to whether or not the introduction of a properly designed side-span stiffening truss, hung from the cables, would give better results, under the conditions of the case.

Mr. La Chicotte H. A. La Chicotte, M. Am. Soc. C. E.—The subject of suspension bridges has not received the same attention from engineers as would be expected from the great advancement which has taken place in recent years in other branches of engineering construction. It is scarcely two decades since the development of American bridges approached present standards, and in this time but one suspension bridge of first importance has been constructed in this country. The demand for this construction has been limited by the rare conditions which warrant its adoption; and, consequently, it is not strange that this subject has been comparatively neglected, and that its literature should be meager and widely scattered. The author's contribution, therefore, is of interest and value to American engineers, as containing much information on this rather abstruse subject not hitherto readily accessible.

It is to be regretted, however, that the author has not made his discussion more general, and made freer use of diagrams and sketches to illustrate his meaning more clearly, inasmuch as the text of the paper is often obscure or ambiguous.

In making such free use of data pertaining to the proposed North River Bridge, and, in fact, basing his discussion of the subject on a special design for that particular location, there is a tendency to limit discussion and direct attention outside the lines suggested by the author. At the outset of the paper the features chosen for special attention are the proper number of hinges to be used in the stiffening truss, and the proper depth of the truss. None of the questions

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raised on these points has been answered directly, although a certain depth was assumed for each style of construction; but nothing is given to show that such assumptions are proper, or that some other depth of truss might not have given better results. In a general discussion it is not so much a matter of importance that a three-hinged truss 140 ft. deep be cheaper than a two-hinged truss 60 ft. deep, or than cable bracing 50 ft. deep; but rather that the foregoing depths are proper in each case, and why. The author has, no doubt, fully investigated this branch of the subject, and will give it in some detail in the final discussion. Mr. LaChicotte

Because of the large amount of labor involved in a technical discussion of this kind, it is hardly to be expected that engineers will do more than engage in what might be termed a negative discussion; that is, where the author's arguments are not convincing, or even when direct issue is taken therewith, a mere statement to that effect should be sufficient without adducing contrary proof.

One of the chief points made by the author in favor of the three-hinged truss is that of maximum grade, this being for the three-hinged truss only half of that found for the two-hinged truss. This is a matter of great importance in a bridge to be used for railroad traffic.

The author's comparison is hardly fair, because he uses the same deflection at the center of the main span for both designs. Under the influence of live load and temperature, the cable deflects quite uniformly in the case of the two-hinged truss, while, with the three-hinged truss, the cable forms a kink at the center of the main span under the restraining action of the two half-trusses. The reaction of the trusses at the center hinge pulls the cable lower than in the other case, thereby increasing the deflection materially for the same load and temperature, and consequently increasing the grade. Approximate computations, based upon the author's assumptions, show this increased deflection of the three-hinged truss to be 46% greater than that of the two-hinged truss for the same load and temperature.

More accurate calculations, based upon proper assumptions, will show a real difference somewhat less than that given above.

The effect is to increase the grade of the three-hinged truss above the value given by the author.

Again, the author's assumption of uniform unit stress throughout the chords of two-hinged trusses cannot be realized in practice. The moment of inertia of the trusses will be nearly constant over the middle half of the span, decreasing toward the end supports, but not so fast as the moments due to temperature and load. The elastic curve in such case will lie intermediate between those for beams of uniform strength and of uniform section. In practice, therefore, the two-hinged truss will not bend in the form of a circle, and the grade

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Mr. LaChicotte

will be less than given by the author. The net result is that, under the same conditions of load and temperature, there is a slight advantage in the ratio of about 7 to 8, in favor of the three-hinged truss.

For finding the stresses produced by the change of length of the cables in suspended stiffening trusses of three-hinged trusses, the author gives the formula:

$$T = \frac{\delta_1 (w + q) L^2}{h l^2}$$

where T = the load per linear foot of truss induced by a deflection δ_1;
L = span of cables, from center to center of towers;
l = " " trusses, " " " " " supports;
w = weight of cables per linear foot of bridge;
q = pull of suspenders " " "
h = versed sine of the cable.

If, for simplicity, the span of the cable and the truss be the same, or $L = l$, and as $(w + q)$ = the dead load of the bridge, the foregoing formula may be written:

$$T = \frac{\delta_1}{h} (\text{dead load}).$$

That is, the load, T, which is simply a portion of the dead load picked up by the trusses during deflection, is quite independent of the length of span or moment of inertia of the truss, and depends only on the total dead load of the bridge and the ratio of deflection to the original versed sine, or dip of the cable.

If $\delta_1 = h$, the truss would carry the whole uniformly distributed dead load except the weight of the cable, and the cable would carry its own weight and the reaction of the trusses at the center hinge. Under these conditions the cables would assume the position of tangents to their original curve, but they are prevented from so doing by their own weight, and because their length is not sufficient to permit it.

The formula is not correct in form, but may give approximate results for comparatively small deflections.

Again, in the application of this formula, exception may be taken to the author's statement that:

"In three-hinged stiffening trusses those stresses which are due to the lengthening of the cables from moving load are of opposite sign to the maximum moving-load stresses occurring at the same time; their neglect is therefore on the safe side, giving maximum stresses considerably too large."

This statement is apt to be misleading, and this instance serves to emphasize the danger of basing a general argument on a specific case involving special conditions and assumptions.

The formula previously given denotes the load imposed on the trusses by a deflection, δ_1, due to elongation of the cables. There is no reason for discriminating between the elongation due to temperature and that due to live load, except as to amount; in other words, it

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matters not how the elongation of the cable is produced, it will cause deflection, and the latter will induce stress in the trusses. The positive and negative moments are not necessarily coincident or equal, as the maximum position may be produced by a short load more or less concentrated, while the maximum negative moment in one half-span is generally produced by a load covering nearly all of the other half-span. Broadly speaking. however, the maximum positive and negative moments of the two half-trusses are about equal and occur simultaneously, when the load nearly covers one half-truss. Mr. LaChicotte

Under this condition, a rise of temperature increases the positive moment and decreases the negative, while a fall of temperature has the contrary effect. The effect of temperature on the moments is due to a change of length of the cables, and any cause which produces a change of length of cable will affect correspondingly the moments in the trusses.

If the supports of the trusses are adjusted so that the latter are free from stress at mean temperature, the maximum stresses in the trusses are due to partial live load only, and are equal and opposite.

If the temperature rises or falls, and produces positive or negative stresses in the trusses, these stresses are to be added algebraically to those produced by live load, in order to obtain the maxima. Therefore, at a low temperature, the maximum or total negative stresses will be larger than the positive, and *vice versa*. Since any elongation of the cables produces positive stress in the trusses, it is necessary, in order that the maximum positive and negative stress be equal, that the neutral condition of the truss be assumed at such temperature and loading that the total upward and downward deflection of the truss, under combined temperature and live load, be equal.

The correctness of the author's statement, quoted previously, depends entirely upon the assumed temperature and loading at which the trusses are neutral or not stressed.

If such assumptions give greater negative than positive stresses, the statement is correct; as any elongation of the cables, whether from temperature or live load, reduces the negative moments or stresses in the trusses; but it likewise increases the positive moments, and therefore, for the neutral condition of the trusses, that condition of load and temperature should be chosen which will make the maximum positive and negative moments, under these combined influences, equal, and the smallest possible.

Another statement, in which the author has not made himself clear, is the following:

"The stresses in the chords of three-hinged stiffening trusses produced by changes in the length of the cables vary with change in the depth of these trusses in the same ratio as the stresses due to an unequally distributed moving load. The chord sections must vary nearly

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Mr. LaChicotte

as the sum of these stresses. The former stresses per square inch of chord section, therefore, are nearly the same, whatever the depth of the three-hinged trusses."

It is quite evident that the author does not convey the meaning he intended, but there is also some doubt as to what that meaning really is. Why temperature stresses should vary with the depth of truss "in the same ratio" as live-load stresses, is not easily apparent. Upon the author's assumption that the two half-trusses remain practically straight for all positions of the cables—a condition approximately realized only with very deep trusses—the chord stress will vary directly as the depth of truss in the same manner (not ratio) as the stresses due to live load. This being so, the next statement of the author follows naturally, viz., "the chord sections must vary nearly as the sum of these stresses," but it must be added that the chord sections vary inversely as the depth of the truss. It does not follow, however, that "the former (temperature) stresses per square inch of section, therefore, are nearly the same, whatever the depth of the three-hinged trusses." Such may be the fact, but the arguments leading up to this conclusion are far from convincing.

It is true that the deeper the truss, the greater the moment tending to produce distortion, but this moment depends upon the rigidity, rather than upon the depth, of the truss. It is impossible, however, that the temperature stress per square inch of chord section should vary directly as the depth and at the same time be "nearly the same, whatever the depth of the three-hinged trusses."

The author's statements, as quoted herein, are too sweeping, and, while he may find some justification for them in the special design with which he deals, in the speaker's opinion, they will not be found capable of general application.

Mr. Hildenbrand.

W. HILDENBRAND, M. Am. Soc. C. E.—In discussing a paper laid before this Society it is generally understood that the discussor either agrees with the author's statements and opinions by corroborating them with data from his own studies and experiences, or that he disagrees with the author by pointing out the latter's errors or supposed errors.

It can readily be seen that Mr. Mayer's paper is of such a kind that no distinct opinion about its merits or deficiencies can be expressed without having preceded such opinion with a profound and time-consuming study. The paper, in the main, consists of the amplification of a certain theory, expressed in mathematical formulas, for proportioning, under given conditions, the dimensions of a stiffening truss for a railway suspension bridge. While the form of the paper may have been specially prepared for its presentation to the Society, there is no doubt that the substance of it is the result of previous studies which extended over many months, and probably years. With-

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out having devoted sufficient time for a thorough study of the paper, the speaker's familiarity with the subject may justify him in saying that the formulas of the theory, as graphically expressed in the curves of moments and vertical forces, seem to indicate that Mr. Mayer has not radically deviated from known theories, and that in the main principles he agrees with other students and writers on the same subject. Mr. Hildenbrand.

The statement has been made that the literature on suspension bridges is very scant, which is relatively true if compared with the numerous books written on the calculation of truss bridges, but there exist quite a number of publications treating the subject of stiffening suspension bridges. Of those with which the speaker is acquainted, the following authors may be mentioned:

Charles Bender, Professors Dubois and Johnson, of American writers; Professors Ritter and Melan, of German writers; and Professors Rankin and Henrici, of English writers.

The theories of the respective authors differ more or less in their line of reasoning, but they have two qualities in common, viz., that all of them are wrong, and that, nevertheless, the results obtained by the given formulas are satisfactory for all practical purposes.

In saying that the theories are wrong, it is not understood that they contain mathematical errors, but that the nature of the problem is so complicated and uncertain as to make an absolutely correct solution impossible. Hence, all the attempted solutions are, at best, only approximations, which are sufficient for practical application, but which do not give real reliable values for the stresses in the different members of the stiffening construction.

This fact may readily be understood if we consider that even in such a simple problem as the calculation of an ordinary truss bridge we must acknowledge that uncertain stresses exist in the web members, of which the direct theory does not take cognizance. Going a step further, and considering trusses which are continuous over two or three openings, we know that the uncertain elements in the calculations have already accumulated so much that many engineers and authors on bridge constructions, like the above-named Charles Bender, have condemned the application of continuous girders as unsatisfactory, uneconomical and not worthy of the immense mental energy needed for making the complicated calculations. While we must admire such elegant solutions of the continuous-girder problem as the formulas of Clapeyron, Mr. Bender thinks that the only good coming out of such calculations is that of a mathematical exercise.

The stiffening truss of a suspension bridge contains, in an aggravated state, all the inherent uncertainties of the continuous truss. It is a truss, continuous over, perhaps, 100 to 150 openings, with flexible supports and uncertain reactions, the flexibility of the supports depending on the flexibility and deformation of a chain or cable, with

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Mr. Hildenbrand.

the curve of the deformation not accurately known. From this description it may be seen that the calculation of a stiffening truss must be based on so many simplifying suppositions that the results can only be approximations to the reality, but never be absolutely correct.

Notwithstanding this fact, the subject of stiffening trusses was considered of sufficient importance to induce engineers and professors to make of it a special study, and it is important enough to deserve the further attention of those who are interested in scientific bridge construction. For this reason Mr. Mayer is to be congratulated for his contribution to the literature on suspension bridges, which should earn for him the thanks of the profession.

The speaker, too, has devoted years of study and practice to the efficient and economical construction of stiffening systems for suspension bridges, but he has long ago come to the conclusion that the importance of the subject has been, and still is, over-estimated. On various occasions the speaker has had opportunity to make what may seem to be a sweeping assertion, that stiffening trusses in a suspension bridge are rather luxuries than necessities. In other words, while a certain rigidity in a suspended floor is agreeable, and, indirectly speaking, may be necessary for practical reasons, it is by no means a vital part of the construction, and, theoretically, it is not necessary for the safety of the structure. This statement is fully and unquestionably true for highway bridges, and it also holds good, to a greater or less extent, in the case of railway bridges. Nearly all the existing suspension bridges give evidence of the correctness of the speaker's assertion, and, among the many suspended structures which have come under his observation, the following may be mentioned as illustrative examples:

A highway bridge, of about 400 ft. span, over the Elk River, near Charleston, W. Va., is suspended from wire cables without any kind of stiffening construction. The bridge accommodates a heavy traffic from the adjoining lumber region, and the oscillations and undulations of the floor, under the moving load of a four-ox lumber wagon, are enough to make a person seasick, but without causing an apparent inconvenience to the travel of vehicles, and certainly without detriment to the strength and durability of the structure. The bridge, as far as the speaker could ascertain, was built long before the civil war; therefore, at the time it came under his observation, it had seen at least thirty years' service, and it was still in an excellent state of preservation.

In 1899 the speaker was engaged to inspect a suspension bridge over the Merrimac River near Newburyport, Mass. This bridge was built in 1801, and, therefore, if not absolutely the oldest, is one of the oldest, structures in the country. It is only a small bridge, of 244 ft. span, and consists of an unstiffened floor suspended from chains made

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of iron links, which probably were forged by an ordinary village blacksmith. It is probable that at the time of its erection, a century ago, this bridge was considered a wonderful structure, and its strength and safety have certainly stood a test with which that of few bridges, even of more modern design, can be compared. Mr. Hildenbrand.

When, some twelve to fifteen years ago, trolley cars were invented, the community owning this bridge did not hesitate to permit the crossing of electric cars, without making any change or addition to the structure, except to provide it with the necessary girder rails. The effect of a trolley car, weighing, perhaps, 12 or 14 tons, on the unstiffened floor was more appalling to the speaker than anything he had ever experienced. Standing at one end of the bridge and seeing a car enter at the other end, it seemed suddenly to vanish from sight, giving the impression that the bridge was breaking down, but in a few moments the car emerged again, and after it had reached the opposite end the bridge assumed its usual shape and appearance, as though it had never been disturbed. Actual measurement showed that the car caused a local deflection of about 20 ins. in one quarter of the span, and a corresponding rise in the opposite quarter. The combination of the depression and rise projected on the car produced the impression that the car was gradually vanishing from sight. The speaker made a careful examination of all parts of the bridge, and found them in perfect condition and of ample strength for sustaining the loads; but, not sharing the unflinching confidence of the Newburyport community, in the judiciousness of a flexible floor to the extent described, he recommended the construction of a light stiffening truss and an additional wire-rope cable for carrying the additional weight. The suggested improvements were executed, and they accomplished the desired effect, but it may be an over-estimation to assume that they increased the safety of the bridge, which, before these alterations were made, had stood all the vicissitudes of the elements, and all the travel for a century.

As to railway suspension bridges, there is at present not a single structure of that kind in existence. The first, and up to this time the only, suspension bridge which has carried railroad trains was the well-known Niagara Bridge, built in 1854 by John A. Roebling. It had a span of 810 ft., and was stiffened by two wooden trusses, which had no regular chords, but consisted only of posts and diagonals. Being a double-deck bridge, the trusses were apparently designed more as separators between the two floors than as stiffeners. However, the floor stringers must have acted as truss chords, because the trusses stiffened the bridge satisfactorily for all practical purposes. The local deflection of the floor under the weight of a passing freight train was observed by the speaker to amount to not more than 15 or 16 ins. After the bridge had been in service for about twenty-eight years, without an

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Mr. Hildenbrand.

accident having occurred to any train, some repairs and improvements became necessary, and, among others, the old wooden truss was replaced by a modern steel truss, designed by L. L. Buck, M. Am. Soc. C. E. It is probable that this truss was calculated in accordance with one of the published stiffening theories, and, as a fact, it stiffened the bridge better than the wooden truss, reducing the local deflection to less than 12 ins. However, it is evident that the truss was too stiff for its strength, because at one time, when a heavy train was passing over the bridge, it happened that the top chord snapped in two with a report like a gunshot. One would suppose that this accident caused some calamity, but it did not. The locomotive engineer naturally became frightened by the loud report and by the imagination of an unknown danger, but he conducted his train across the bridge without delay or impediment, and other trains followed without knowing or noticing that anything was wrong. It took some time to repair the damage, and as long as the break existed the bridge was theoretically without stiffening, but the practical effect on the deflection of the floor was hardly noticeable. The traffic went on as usual, and, if the break had never been repaired, it is probable that the public would not have considered the bridge less serviceable than it was before. It may be mentioned that, after the broken chord was repaired, all the diagonals were loosened. In other words, the truss was made more flexible and the bridge less stiff, which answered the practical purposes equally well and avoided all accidents of that kind in the future.

Another striking example of a practically very rigid structure, which theorists had condemned as being insufficiently stiffened, is furnished by the old Cincinnati Bridge, built in 1865 by John A. Roebling. The floor of this bridge, which has a span of 1 056 ft., was stiffened by two light iron trusses, 10 ft. high, which were erected in 30-ft. sections, connected with each other with flexible joints. Mr. Charles Bender, whom the speaker has mentioned several times in this discussion as a man of unusual mathematical knowledge, and foremost in the profession as a scientific bridge constructor, was very severe in his criticism of the Cincinnati Bridge. He considered the trusses to be mere dead weight, and, therefore, worse than useless, because, on account of the flexible joints, these trusses could not act as stiffeners before the floor had deflected 30 ft. It is not likely that a man of such profound knowledge as Mr. Bender made an error in his calculations, but, nevertheless, he was mistaken. The bridge never deflected more than 6 or 8 ins., and appeared to be very rigid even under trying circumstances, such as the crossing of a large drove of cattle. During its long service, extending over thirty-two years, there was never a complaint about too little rigidity, and the traveling public was never inconvenienced or alarmed by too vehement undulating motions.

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That the trusses, notwithstanding Mr. Bender's calculations, were instrumental in producing the stiffness of the floor, became manifest when the speaker was rebuilding the superstructure of that bridge in 1897, and when, preparatory to erecting new trusses, the old trusses were loosened from their connections with the floor beams. The floor became at once much more flexible and shaky than it was formerly, so that a number of people, accustomed to the former rigidity, and erroneously ascribing the unusual motion to some weakness in the structure, became alarmed, and temporarily used other means of communication between the two cities of Cincinnati and Covington. Mr. Hildenbrand.

The speaker could cite more cases of suspension bridges having no, or insufficient, stiffening systems, and which have for years safely accommodated their respective traffic, but the four examples mentioned may suffice to corroborate the speaker's opinion that, while it is desirable to provide a well-constructed suspension bridge with a proper stiffening truss, the latter does not form a vital feature in the existence of the bridge.

With the statement of this fact, the speaker does not wish to discourage the scientific research for establishing rules for stiffening suspension bridges, because the construction of a stiffening truss will be as much a demand for every modern suspension bridge as springs and cushioned seats are for a railway car. On the other hand, the speaker does not wish to frighten engineers of less mathematical training by magnifying the importance of a stiffening truss of which the theory is one of the most uncertain and most complicated problems in the science of applied mechanics. It is not to be wondered, therefore, that the opinions of engineers and authors differ as much about the general construction of a stiffening truss as they do about the development of the respective theories. Some engineers advocate continuous and shallow trusses; others, high and center-hinged trusses; others, trusses fastened at the ends and acting partially as cantilevers. Each method requires a different theory, and, while none of these theories is absolutely correct, each author has good reasons to assume that his theory will give satisfactory results. A serious mistake can hardly be made, except if too stiff a truss be constructed. Too much flexibility may perhaps cause some inconvenience, but it will never be harmful. From what is known of Mr. Mayer's design of a single-span North River bridge which formed the basis for the theory and calculations presented in this paper, the speaker would not imitate the same construction; but he is also satisfied that a truss calculated on Mr. Mayer's formulas will fully answer all requirements.

If the speaker's theory on stiffening trusses differs from that of Mr. Mayer, he does not claim that it is better or more correct, but he has the advantage of being able to point to a practical demonstration of that theory in the reconstructed Cincinnati Bridge. This is the first bridge

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Mr. Hildenbrand.

in the country provided with an adequate stiffening truss according to modern principles and requirements. It was calculated and proportioned on the speaker's design and theory, with results highly satisfactory to the owners of the bridge and to himself. The bridge has now been in use for four years, and it is more rigid than the majority of so-called "rigid" bridges, which term is frequently applied to truss and cantilever bridges in comparison with the term "swing" bridges as applied to suspension bridges.

In regard to Mr. Mayer's assertion that all possible stiffening systems for long suspension bridges other than trusses or bracing between parallel cables or chains are "now generally conceded to be inferior," the speaker would like to know the authorities for this statement, and on what grounds Mr. Mayer considers them as experts. This question is the more justified because all existing large suspension bridges are partially stiffened by other methods than mentioned above, and, inasmuch as these bridges are actually stiff, the speaker would suppose that no opinion advanced in contradiction of this fact would deserve to be called "expert" opinion before its superiority had been demonstrated in practice. As there are no such examples in existence, the speaker fails to acknowledge Mr. Mayer's sweeping assertion.

The speaker would like to ask what Mr. Mayer calls a "properly designed" cable. If a "properly built" cable had been mentioned, the speaker would agree with him, because, in the practical construction of a cable, serious blunders may be made, while hardly any mistake is possible or probable in the design of a cable. Unless Mr. Mayer refers to a cable of some fancy construction, the design of a plain wire cable is extremely simple, and not subject to much variety.

The speaker notices that Mr. Mayer advocates the adoption of high unit stresses for cables and stiffening trusses. As to the cables, which are the most important parts of a suspension bridge, the backbone of the structure, so to say, the speaker prefers to be a little conservative, but he fully agrees with Mr. Mayer that unit stresses of 30 000 lbs. per square inch for the stiffening trusses are well justified. Considering that the maximum stresses occur but rarely, and that stiffening trusses are altogether only secondary parts in a suspension bridge, it would be a waste of money to construct them with the same factor of safety as one is accustomed to use in truss bridges. While the rupture of any part in an ordinary truss is fatal to the bridge, it has been demonstrated in this discussion that a break in a stiffening truss is not followed by a calamity, and not even by an inconvenience. In 1897 the speaker had an opportunity to submit a design, for a railway bridge across the North River, to a committee of engineers, appointed by the President of the United States, for examining the feasibility of such a project. In the calculation of the stiffening truss, the speaker had adopted the conservative unit stress of 18 000 lbs. per square inch, but

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the Committee instructed him to modify his calculations by reducing the unit stress to 15 000 lbs., which he did, though under protest. The consequence was that the cost of the speaker's design was higher than that of another design, for a cantilever bridge, which was to be compared with a suspension bridge. With a unit stress of 18 000 lbs., the cost of the two designs was about the same, but if the speaker had been permitted to adopt 30 000 lbs., as suggested by Mr. Mayer, the suspension plan would have been, in point of economy, far ahead of the cantilever plan. Mr. Hildenbrand.

It is gratifying to the speaker, therefore, that times have changed, and that Mr. Mayer's opinion, which at that time was on the side of the President's Committee, has come to agree with the opinion which the speaker has held for many years.

L. L. Buck, M. Am. Soc. C. E.—In reference to the question of stiffening trusses: When the speaker rebuilt the suspended structure of the Niagara Railroad Suspension Bridge, the moving loads per foot passing over it were about 1 200 to 1 400 lbs., and the trains were hauled by a medium-weight shunting engine. The structure had to be erected without interrupting traffic over the bridge, and, consequently, it was necessary to give the trusses about the same depth as the old ones. Mr. Buck.

The speaker would have liked to decrease the depth somewhat, but that was out of the question. The greater the flexibility of the truss the less the stress upon its members. This flexibility would ordinarily be obtained by giving the truss a depth that would allow that flexibility with given unit stresses. With the depth as required, it was necessary to obtain the flexibility by slacking off the truss rods, as each truss rod was provided with a turnbuckle having right and left hand threads. By first bringing the turnbuckles up to a bearing, and then turning the turnbuckle back about half a turn, it gave $\frac{1}{8}$ in. of slack, which afforded the flexibility required.

Some years after, during August, when the temperature was about maximum, the foreman noticed that these rods were slack, and, thinking that they ought to be drawn up tight, went to work and screwed them all up. They had formerly been adjusted at mean temperature, in which case the temperature would produce no stress on the members at that temperature, and as the whole change or deflection between the extremes of temperature was 2 ft., of course the truss would only have to bend 1 ft. each way from the mean; but tightening up these truss rods during the hottest weather required a bending of 2 ft., enormously increasing the temperature stresses. At the same time, the loads per foot had more than doubled. One cold morning in the winter, when the upper chord had a very heavy tensile stress upon it from temperature, a heavy coal-train, of ten cars, with the heaviest loads on each car, ran out on the bridge from the New York end, and produced a

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Mr. Buck. very heavy stress on the upper chord at one-third the distance from the other end, and the chord broke. The train went on across the bridge, and merely closed up the break, and it was fully as capable of resisting a compressive stress as ever.

The speaker cannot quite agree with Mr. Hildenbrand's suggestion that the stiffening structure is not important in the suspension bridge, but thinks it is of vital importance. In the first place, with excessive deflections, the motion is very disagreeable to passengers riding over the bridge, and, with such excessive undulations, the cables and other members would rapidly wear out.

There is another question as to the depth of the stiffening truss that is not very often considered, and that is the amount of versed sine of the cable. If the versed sine is large, the truss can be deeper than if the versed sine is smaller, in proportion to the span. With the larger versed sine there is not so much vertical change in the cable between extremes of temperature.

A steel tower has the effect of decreasing the amount of deflection between the extremes of temperature, as when the cables lift up, the tower is shortened somewhat—about 1 in. in 100 ft.—and *vice versa*, thus decreasing the change.

Mr. Duryea. EDWIN DURYEA, Jr., M. Am. Soc. C. E. (by letter).—This discussion is confined to a comparison of the three-hinged with the two-hinged stiffening truss, and is an attempt to show qualitatively, rather than by figures, that the function of a stiffening truss can be performed much more economically by the former.

The primary object in any bridge structure—the reason for its existence—is to furnish a convenient and unobstructed roadway for traffic. Everything about the structure should be subservient to this end. Incidentally, the structure must also be of sufficient strength to carry safely all loads likely to come upon it.

Next in importance to making a bridge convenient and safe for its traffic, however, is the attainment of these objects for the least practicable cost. This, in many cases, is even of equal importance with the primary object of the structure. If the bridge is to be built by a private company, no bridge of any kind will be built unless its estimated cost is so small that the net revenue expected will yield a profit on the investment. This consideration does not have the same weight when bridges are to be built by cities, as usually no direct revenue is then expected. Besides, cities do not seem to consider economy in expenditures so important or necessary as do private corporations. Even in the case of city bridges, however, the permissible cost of the bridge is supposed to have a reasonable relation to the direct and indirect benefits which will be produced by it. It may even happen, if the credit of a city is poor or its limit of debt-incurring capacity nearly reached, that the bridge cannot be built unless its cost can be kept within a

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certain fixed amount. The question of cost, therefore, may be of extreme importance, even for bridges built by cities, and is always so for bridges built by private companies. Mr. Duryea.

The suspended superstructure of the most generally approved type of stiffened suspension bridge consists of three principal parts: The cables, the floor or platform with its suspenders, and the stiffening trusses. The function of the cables is to furnish the carrying power from tower to tower; that of the platform and suspenders is to furnish an approximately horizontal surface, attached to the cables, on which the moving loads may cross the bridge; and that of the stiffening trusses is to prevent too great departures from the approximately horizontal shape of the platform by the distorting effect of train loads covering a part of the span only.

If the moving loads are very light, or are of such a nature that the existence of steep and changing grades is not important, the bridge may consist of the cables, platform and suspenders only, or even in extreme cases of only the cables and a platform laid directly upon them. When the bridge is to carry trains, however, it is absolutely necessary that the grades of the tracks shall not, under any conditions, exceed some chosen limit of steepness. While the cables possess all the necessary strength, they have practically no stiffness or resistance to bending. Both cables and platform are flexible, tending to deflect downward throughout the length of the train and upward elsewhere. This tends to keep the train always in a hollow out of which it is trying to climb, or to make an up-grade traverse the structure in a wave ahead of the train. The depth of this hollow and the steepness of this grade will be greater as the relative proportion of train load to dead load increases.

From computations once made by the writer, the extent of the deformations of an unstiffened suspension bridge would be as follows, for the conditions given below:

Span from center to center of towers = 2 850 ft.; sag = one-eighth of the span.

Total loads on bridge:

Dead load = 29 000 lbs. per foot, over whole span.
Moving load = 15 200 lbs. per foot, 1 050 ft. long.

With the moving load beginning at the center of one tower and extending 1 050 ft. toward the middle of the span, the deflections of the cables from their position when no moving load is on the bridge would be:

Maximum downward deflection = 19.5 ft. at 525 ft. from loaded tower.
Maximum upward deflection = 17.1 ft. at 2 025 ft. from loaded tower.

These points are 0.184 and 0.711 of the cable span, respectively, equal to about two-elevenths and eight-elevenths of the span, or,

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Mr. Duryea. roughly, to the quarter-points. The deflections at the quarter-points would be about 18.5 ft. down and 16.6 ft. up, respectively. At the center of the span the cables would rise 7.9 ft. The distorted cables would cross their former position at a point about 1 215 ft. from the center of the loaded tower, or 165 ft. beyond the head of the train load. The up-grade ahead of the train in this position would be about 5 per cent.

The span, sag, weights, etc., are practically the same as those assumed by Mr. Mayer for the three-hinged truss design of his paper. An approximate allowance was made in the computations for the stretch of the cables and for the saddle run.

The foregoing data as to distortion give a concrete idea of the cable movements which would occur if the bridge were of full weight and yet entirely unstiffened, the function of the stiffening truss being to limit these movements to a very small amount. These data are of no practical value, though use could be made of the method to obtain the values of the bending moments which must be resisted by the structure if the cable distortion is to be confined to chosen limits. The method is a trial one, however, and very tedious, especially when complicated by temperature changes and the necessity of taking the train load in a large number of positions. Its use is impracticable in comparison with the more exact, positive and rapid method originated by Mr. Mayer.

With the cables unstiffened, distortions would occur in the grade line so large as to interfere with the running of trains. These distortions may be confined within limits which will not interfere with trains by any means which will add sufficient stiffness to the structure. The stiffness desired may be furnished by any form of stiffening truss, one with three, two or no hinges. The object sought is stiffness, not carrying strength, and it is evident that the form with no hinges must be stiffer than the two-hinged, and the two-hinged stiffer than the three-hinged, form. If there were no conflicting considerations, therefore, and no other condition to provide for than the prevention of distortion under partial loads, the desired degree of stiffness could be added to the cable most economically by means of stiffening trusses having no hinges, and least economically of all by means of stiffening trusses having three hinges.

A conflicting condition exists, however, which changes the problem entirely. At some fixed temperature, in conjunction with some special condition of loading, the stiffening truss does indeed have to perform only the duty of adding stiffness to the cable. Whenever either temperature or load changes, however, other influences than those of unsymmetrical distortion begin to act on the stiffening truss. With any change of either temperature or loading from the standard, the length of the cable changes and also the cable span. The span always decreases at the same time that the cable lengthens, and *vice versa*, so

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that changes in cable length always produce a much larger change in the sag. The stiffening truss is connected with the cables by suspenders which are practically inextensible, and it must, therefore, rise and fall as the sag of the cable changes. The stiffening truss, of whatever number of hinges, is fixed vertically at each tower. When the temperature or the loading decreases, the cable shortens, and rises because of the decrease in sag. This rise, which is greatest at the middle of the span, evidently bends the trusses upward. Unless the trusses are to a certain degree flexible, this upward bending may cause over-stresses or even rupture in their chords. Mr. Duryea.

When the sag of the cable increases, because of increase in the temperature or the load, the stiffening truss, except where its ends are fixed at each tower, is evidently allowed to deflect downward. This downward deflection must produce upward reactions at each tower. The stiffening truss, the real function of which is only to add stiffness to the cable, has then become to some degree a load carrier also, carrying to the towers part of the load which was formerly carried by the cable.

The span of a suspension bridge is always comparatively long, too long for any simple truss. If the stiffening truss is made stiff, it will, so to speak, try to carry some of the load over the long span, and its chords will be over-stressed or ruptured in the attempt. This danger can be prevented only by making the truss so flexible that it will bend easily under the load coming upon it, thus, through the suspenders, throwing all the load over a certain amount back on the cables.

There is no practicable way to prevent changes in the sag of the cable. The changes in sag due to temperature changes are independent of the section of the cable. Even the variations in the sag caused by changes in the load could be materially lessened only by such large increases in cable section as would neutralize all the economical advantages of a suspension bridge. Therefore, it may be considered an established fact that upward and downward movements must be provided for in stiffening trusses, and that the only way to do this is to in some way make the trusses flexible so that they can follow the movements of the cable without injury to themselves.

There are two distinct ways in which this flexibility may be provided. In both no-hinged and two-hinged stiffening trusses, the trusses have about equal flexibility in all parts, bending in smooth arcs from tower to tower, and the form of the cable after the change in sag is still a single smooth curve, with the tangent to its middle point still horizontal. In the three-hinged stiffening truss, however, the flexibility is all concentrated in the hinges, the trusses elsewhere having but little more flexibility than ordinary simple trusses. The form taken by the three-hinged truss after bending will be that of two approximately straight lines, meeting in an angle at the middle of the

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Mr. Duryea. span. The form of the cable, which before bending was one parabola with its tangent at the middle of the span horizontal, will be changed to two parabolas intersecting each other at an angle in the middle of the span. The change in form of each half-cable is very small, as the bend at the middle hinge concentrates in itself most of the total change in the curvature of the cable. The temperature stresses in each half-truss, therefore, are small—too small to exercise any important influence on the economical layout of the truss.

It has been shown that a two-hinged stiffening truss must be flexible throughout its whole length, while a three-hinged truss need be flexible only at its hinges. The only way that a truss can be made flexible, except by hinges, is by keeping its height low. It is a well-recognized fact that a low truss cannot be economical. In order that a truss be economical in weight and cost, its height must bear a certain relation to its span, and must be greater at the middle of the span than at its ends. Any large departure from this ratio (which is about one-eighth for the longest simple trusses yet built) must result in a large increase in the weight of the truss and in its cost. It is evident that a truss having two hinges only, one at each tower, will have its greatest stress from cable movements at the middle of its own and the cable span, where the change in sag is the greatest. This is the very point where the truss should, for economy, be the highest and least flexible, and, therefore, it is apparent that the two-hinged form of stiffening truss is one which is inherently opposed to economy.

The three-hinged truss, however, is free to bend at the middle of the span. Most of the change in shape of the cable occurs at the same point. At the quarter-points of the cable span (which are also the middle points of each half-truss) the change in form of the cable is small, and the half-trusses, therefore, can be made stiff and high at their middle, thus conforming to the main principle of economy, and effecting a large saving in weight and cost over a low truss.

The greatest advantage of the three-hinged over the two-hinged stiffening truss lies in this possibility of a large saving in cost, and this feature will be referred to again. The three-hinged design has other important advantages, however. The existence of a middle hinge fixes at the hinge the point of no moments for all positions of the moving load, thus making the structure one which can be treated by statics instead of only by its elastic properties. While the elastic properties cannot be entirely neglected, even with the three-hinged form, they have a small influence only; the computed maximum distortion of the 1 400-ft. half-span of the stiffening truss for the Hudson River Bridge design being only 2.82 ft., or only about the same proportion of the span as the camber of a simple truss. This practical removal of the structure from a class which must be investigated by elastic properties to one where the principles of statics can be applied

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is in itself a great advantage, and eliminates many uncertainties from the problem. This will be recognized by engineers and by all but enthusiastic mathematicians. Mr. Duryea.

Only two objections to the middle hinge can be made—that it will not work as a hinge, and that harmful bending stresses will be caused in the cables at the middle of the span. As to the former, it should be remembered that the changes due to variations of temperature are very slow in their action, and even the changes caused by moving loads are comparatively slow in such long spans. Such a hinge has been studied and re-drawn several times, and detailed plans for it have been drawn out. There seems no more reason to apprehend a failure in its action than in the end pins of any carefully-designed long span. The jar of trains would of itself tend to develop the hinge action.

As to bending stresses in the cables at the middle hinge, some will certainly be produced. If their existence is recognized, however, means can be taken to prevent most of them and to limit the others to a small amount. All suspension-bridge cables possess, so far as concerns direct stresses, an excess of section at the middle of the span, and the total bending stresses here can by proper means be kept so small that a small excess of section will still remain. This point will not be entered into further, except to say that studies and detailed drawings have been made which leave no doubt open as to this conclusion.

The three-hinged truss possesses two other advantages in addition to those already mentioned. In cases where limiting grades may occur, the three-hinged type, as shown by Mr. Mayer, will give a maximum grade less steep than the two-hinged type. The three-hinged design also allows a considerable saving of wind metal in the chords. As the chords are cut at the middle of the cable span, their span for wind loads is the same as for vertical loads, or only one-half the span of the cable. The wind reactions of the trusses at the middle hinge go to the cables, which carry them to the tops of the towers. The section of the tower legs is increased somewhat by this, but the net saving from halving the wind span is large. As already mentioned, careful studies of this middle hinge have been made by Mr. Mayer, and detailed drawings of it have been made.

Two points in Mr. Mayer's design for a Hudson River suspension bridge, not connected with the stiffening trusses, are worthy of special mention as innovations. The first is his contention—now being recognized as justified—that the cradling of the cables is unnecessary, as the raising of the weight during its side motion will soon neutralize the pressure of the wind. The cradling of the cables causes many complications, especially in the details of the anchorages, and its avoidance will allow of much simplification and of a material saving in cost.

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Mr. Duryea. The other novel feature is his use of any convenient angles for the back cables (so long as they are not steeper than the angle of the main cable span at the towers), and yet using on the tops of the towers ordinary saddles on rollers, without any of the complications of a non-continuous cable. Past practice has always required that the angles of the cables should be equal on the two sides of each tower. It is easy to see, however, that so long as the saddles remain movable (and it is entirely practicable to so design roller nests that they will remain movable), no side pull of any greater amount than the friction of the rollers can be put on a tower. The only feature which must be guarded against, when unequal angles are used on the two sides of a saddle, is the slipping of the saddle under the cable, and this can easily be provided against by some form of local attachment. The great advantage gained by using any angle for either back cable is, of course, the free choice of anchorage sites which is thus made possible.

The saving in cost by the use of three-hinged stiffening trusses will now be reverted to.

It has already been stated that the two-hinged stiffening truss must necessarily be much heavier than the three-hinged truss. Mr. Mayer, in his paper, estimates this excess of weight as 8 500 lbs. per foot of bridge, or that the weight of the two-hinged stiffening truss is nearly double that of the three-hinged truss. At $100 per ton, which is probably too low, this excess of steel amounts to $425 per foot of span, or to $1 190 000 for the 2 800-ft. span of the stiffening truss. This, however, is only a small part of the extra cost, as will be seen below. The data given are abstracted from a thorough investigation, by the writer, in 1899, of Mr. Mayer's completed design for a Hudson River Bridge.

Of the cable's working strength, 16.6% (70 000 lbs. per net square inch of wire) is required to carry the cable's own weight.

One square inch (net wire) of the cable has a working strength sufficient to carry only 18¾ lbs. per foot of span in addition to its own weight.

One ton per foot of span of the external dead load (dead load in addition to the weight of the cables) requires 106.7 sq. ins. of net cable wire to carry it.

The total net section of the eight cables is 1 805 sq. ins., and their total weight between anchorages is about 19 000 tons of wire. This is equivalent to 10.52 tons of cable wire per net square inch of cable, or to 1 120 tons of cable wire per ton per foot of span of the external dead load.

In a similar manner, the two anchorages were found to contain about 2.4 tons of anchorage steel per square inch of cable, or about 260 tons of steel per ton of external dead load per foot of main span. The New York anchorage might have to be artificial, in which case it would require about 28 cu. yds. of concrete and masonry per square

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inch of cable, or 3 000 cu. yds. per ton per foot of main span of the external dead load. Mr. Duryea.

One tower contains in its four legs, exclusive of all lateral connections, etc., about 4 600 tons of steel. The dead-load stresses in a tower leg comprise about 65.2% of the sum of all the stresses; therefore the dead-load metal in the legs of one tower is 3 000 tons, or 6 000 tons for both towers. The total weight per foot of main span of the suspended structure is 14 tons, of which 3.2 tons is the weight of the cable wire, and the remainder, 10.8 tons, is the external dead load. The total cable needed to carry itself and this external dead load is 106.7 sq. ins. × 10.8, or 1 150 sq. ins. This gives a weight of steel in the tower legs of 5.22 tons per square inch of dead-load cable, or 555 tons of steel in the tower legs per ton per foot of main span of external dead load.

The proportion of the tower foundations due to dead loads may be taken to be the same as that of the tower legs, or 65.2% of the total. The proportion of the foundation cost due to each ton per foot of main span of external dead load is then equal to 65.2% ÷ 10.8, or to 6.04% of the total cost. The total cost of the tower foundations, even at a low valuation, would be more than $3 000 000, and the cost of the tower foundations per ton per foot of main span of external dead load may be conservatively estimated at $200 000.

The various items are collated in Table No. 1, and given a money value.

TABLE No. 1.—Total Cost of Structure per Ton per Foot of Main Span of External Dead Load.

1 ton per foot of span extra weight of stiffening truss, or 2 800 tons extra of steel	at	$100 =	$280 000
Extra cost produced in other parts of the structure by the above extra weight:			
1 120 tons, cable wire	at	$200 =	224 000
260 tons, anchor steel	at	100 =	26 000
555 tons, tower steel	at	100 =	55 500
6.04% of tower foundation			200 000
			$785 500
3 000 cu. yds. anchor masonry	at	$10 =	30 000
Total, if artificial anchorage is used			$815 000

The 8 500 lbs. per foot of span, which Mr. Mayer estimates would be the excess weight of a two-hinged over a three-hinged stiffening truss, therefore, by the foregoing calculation, would add $3 460 000 to the cost of the bridge. The methods used in the foregoing comparison are approximate only, but close enough for such a purpose. The

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Mr. Duryea. resulting difference in cost is too small rather than too large, as, in addition to the prices used being somewhat low, several items which would help to increase the difference have been omitted from the comparison altogether.

It seems well to say that Mr. Mayer's design for a bridge over the Hudson has been investigated thoroughly, and in a very critical spirit. In 1899, when it seemed quite probable that the bridge would really be built, Alfred Noble, M. Am. Soc. C. E., was retained by the financiers who proposed to build it to make a report on the practicability and correctness of Mr. Mayer's design, as shown by the detailed plans. Mr. Noble selected the writer to make the investigation, and he, with several assistants, spent about six months in doing this work. All data and assumptions were examined carefully for correctness, reasonableness and degree of approximation; every formula used, without exception, was verified rigidly for correctness and for degree of approximation; and the stresses in every member, also the sections and weights, were calculated independently. Nothing was accepted on trust, neither assumptions, formulas nor stresses. On the conclusion of the investigation the tabulated results, with a report by the writer, were submitted to Mr. Noble, and he, after considerable time spent in an independent personal verification of the formulas and methods, made a report to the financiers by whom he had been retained, favorable to the design.

The writer had already had some opportunities to study the design of two-hinged stiffening trusses, through his employment on the New East River Bridge, and his belief in the superior advantages of three-hinged trusses depends, not on the approximate comparisons of Mr. Mayer's paper (which he has yet had no time to study), but on studies of completed designs.

It is worthy of inquiry why so many engineers seem inclined to avoid the three-hinged and to favor the two-hinged type. The reason seems to be either a misapprehension of the general principles involved, or a fear that it may be impracticable to design the details of a hinge so as to guarantee its successful action. As to the former, the only object for which a stiffening truss is ever used is to furnish stiffness; stiffness cannot be secured economically by any other means whatever than by making the truss of considerable height, and the truss cannot be made high unless a hinge is introduced at the middle of the span.

The other argument against three-hinged stiffening trusses—that the action of the hinge may prove unsatisfactory—is an advocacy of the abandonment of a correct general principle because of fear of the impracticability of a detail. In a completed structure, details, of course, are as important to its safety as are layouts or general principles. In a design, however, each should be considered in its proper

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order, the general principles first—and any element chosen and fixed in compliance with a general principle will very seldom have to be discarded because no practicable detail can subsequently be found to carry it out. Mr. Duryea.

Exceptions to this rule are found occasionally, but the three-hinged truss is not one of them. Such an exception exists in the use of braced chains to perform the combined functions of cables and stiffening trusses, as described in Mr. Mayer's paper. In this case the suspended arch, though very good in its general principle, loses all its economy because of the great weight, complexity and untried nature of its details, and because of the necessity for a separate provision for wind stresses.

A familiar instance of the abandonment of a general principle from fear of the impracticability of details is the use of a truss instead of a tie to limit the upward motion of the rear cables of a suspension bridge, when the rear cables carry suspended from themselves the weight of the platform and its moving load. It seems plainly evident that the efficiency and economy of the tie and anchorage over the beam for preventing upward motion is very great, yet the use of the tie has been generally regarded with disfavor. In this case, as in the case of the three-hinged stiffening truss, the apprehension as to details is unwarranted. Careful designing will remove all objectionable features and allow the use of the correct general principle—the tie—with a considerable gain in efficiency and a considerable saving in cost.

While the saving in cost possible from using a three-hinged truss is the main advantage gained, several other advantages have been mentioned. It is less subject to distortion than the two-hinged truss, and its stresses, therefore, can be calculated with more accuracy and certainty; it is much stiffer, and, therefore, will carry a larger proportion of the loads to the towers than will the two-hinged truss, thus relieving to a greater degree the loads borne by the cables; and the limiting grades, when they occur, are less steep than with the two-hinged truss.

The saving in cost which results from using a three-hinged instead of a two-hinged stiffening truss is not secured at the expense of any sacrifice of efficiency, but, instead, is accompanied by a material improvement in the efficiency of the structure. The three-hinged truss gives a structure which is not only as good, but even better, and at the same time saves at least $3 500 000 in the cost.

JOSEPH MAYER, M. Am. Soc. C. E. (by letter)—The discussion of this paper has called attention to the large difference between the deflection at the center of the span of two-hinged and three-hinged stiffening trusses of suspension bridges. The writer neglected this difference in his previous calculations, under-estimating its Mr. Mayer.

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Mr. Mayer. amount. He calculates the center deflection of a suspension bridge with 2 800 ft. span and two-hinged stiffening trusses to be only 71% of that of the same span with three-hinged trusses. He has, however, also neglected another fact which is in favor of three-hinged trusses, and which nearly counterbalances the effect of the former difference on the maximum grade in the two kinds of bridges.

He has compared the maximum grade of the two kinds of bridges when they are unloaded and are at minimum temperature, assuming the lowest position of the track to be horizontal under full uniform load. This comparison neglects altogether the grades produced by the partial loading of the bridge. If the moving load is in the center of the half-span, it produces down grades toward the center of the loaded half-span and up grades toward the center of the unloaded half-span. If the grades produced by the bending of the stiffening trusses are considered separately from those due to the lengthening of the cables, it is found that in the bridge with the two-hinged, shallow trusses of the paper the down and up grades each amount to 1.25 per cent. In the bridge with the three-hinged, deep trusses these grades amount to 0.71 per cent. If the track, for all conditions of loading and temperature, is required to remain above the horizontal, then the bridge, with two-hinged, shallow trusses, must be adjusted so that the grade at the ends of the unloaded bridge at minimum temperature is $1.25 + 0.56 + 1.00\% = 2.81$ per cent.

In this, 0.56% is the down grade at the end produced in a level bridge by the stretch of the cables due to the moving load when it is at the center of the half-span; and 1.00% is the down grade at the ends produced by the rise of temperature from minimum to maximum. The maximum grade at the ends will then be $2.50 + 1\% = 3.5$ per cent. This will occur when the distant half-span is loaded at minimum temperature.

The bridge with three-hinged, deep trusses must be adjusted so that the grade at the ends at minimum temperature for the unloaded bridge is $0.71 + 0.41 + 0.74\% = 1.86$ per cent.

In this, 0.41% is the down grade at the ends due to the stretch of the cables produced by the moving load when it is at the center of the half-span; and 0.74% is the down grade at the end produced by the rise of temperature. The maximum grade at the ends will then be $1.42 + 0.74\% = 2.16$ per cent.

The difference between the maximum grades of the two bridges is 1.34%, which is larger than the difference between the maximum grades given in the paper. The ratio between the maximum grades of 2 800-ft. spans with two-hinged and three-hinged trusses, however, is smaller than given in the paper. These maximum grades, in both cases, can be reduced by allowing down grades from the ends toward the center, under extreme conditions of load and temperature.

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Mr. Mayer.

The writer's calculation of the stresses in three-hinged trusses, due to changes in the length of the cables, has been criticised as giving results which are too small, while it is stated in the paper that the results are too large. Undoubtedly, the effect of the lengthening of the cables is to turn the trusses around the end hinges and drop them in the center, as shown in Fig. 19.

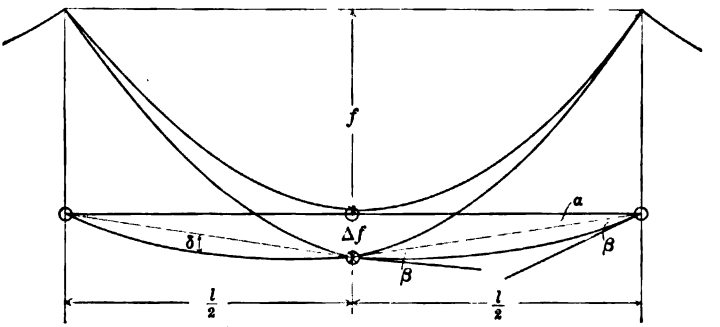

FIG. 19.

After the amount of the drop, Δf, of the trusses at the center of the span is known, it is extremely simple to calculate the angle formed by the cables at the center of the span, if the assumption is made that the trusses of each half-span remain straight after the drop has taken place. This angle is equal to 2 α, if α is the angle of motion of each stiffening truss, and tan. $\alpha = \frac{2\,\Delta f}{l}$. If H is the horizontal component of the stress in the cables, then the upward pull exerted by the cables at the center of the span is H tan. 2 α.

Assuming the trusses in each half-span to remain straight, the writer has proved* that the cables which were parabolas before the drop at the center will be parabolas after the drop, but the parabola of each half-span after the drop will not be a continuation of the parabola of the other half-span. If the cables are parabolas, their loads are uniform over their length. Each half-span of the stiffening truss, therefore, is subject to a uniform load over its length and a given reaction at one end, namely, the center of the span. To produce equilibrium there must be an equal reaction at the other end of the half-span of the truss; and the stresses produced in the trusses, in consequence of a given drop at the center, can easily be calculated. They are evidently proportional to the tangent of the angle formed by the two halves of each cable at the center of the span, as the upward pull of the cables at this point (which produces the stresses here considered) is proportional to the tangent of this angle. Thus far, the writer's argument seems to be

* *Engineering News*, November 7th, 1901.

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Mr. Mayer. accepted. But, after this, the differences between his views and those of his critics arise. What is the effect of the elasticity of the stiffening trusses (which has thus far been neglected) on these stresses?

The stiffening trusses, under the conditions assumed, will bend downward. This will change the shape of the cables, and the angle formed by the two halves of each cable at the center of the span will be decreased. This decreases the upward pull of the cables at the center of the span and therefore decreases the consequent stresses in the trusses. The writer's critic asserts that the elasticity of the truss and its consequent bending has the opposite effect. He bases this statement on the fact that the actual curve of the trusses is intermediate between the straight trusses of the half-spans and an arc of a circle extending from one tower to the other, and that, therefore, the actual stresses in the trusses must be intermediate between the stresses corresponding to the two different shapes. This is evidently true. But the stress in the trusses corresponding in shape to the two straight lines is 0. The inference, therefore, which can be correctly drawn from the foregoing fact is that the actual stresses in the trusses are somewhere between 0 and the stresses which would occur if the truss did bend down in the arc of a circle. The inference drawn by the writer's critic, that the stresses are intermediate between those calculated by the writer and those of trusses without a center hinge is not justified. The writer has calculated the deflection, δ, Fig. 19, due to the temperature stresses alone, in order to determine from this the diminution of the angle 2α of the cables at the center of the span. This diminished angle is $2(\alpha - \beta)$. The diminished upward pull of the cables at the center is $H \tan. 2(\alpha - \beta)$.

The reduction of the temperature stresses due to this cause is 6% of the stress previously found. The calculation of the writer's critic is based on assumptions which do not agree, even approximately, with the facts. The moment of inertia in a properly designed three-hinged truss is from ten to fifty times as much at the center of the half-span as at the end or the center of the span. A calculation based on a constant moment of inertia, therefore, must be of doubtful value.

By another critic the objection is raised that the stresses should be calculated, not only for the lengthening and shortening of the cables due to changes of temperature, but also for the lengthening and shortening due to changes of load.

The bridge should be adjusted so that the stresses due to these causes are the smallest possible. This will be the case if the cables are parabolas, without any angle at the center of the span, for the mean condition of load and temperature. The writer has stated that the moving load assumed was 1 000 ft. long. Only those stresses, from change of length of cables, are wanted which occur together with maximum stresses of the same sign due to moving load. These max-

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imum stresses in the case of a truss of 2 800-ft. span require nearly the whole moving load to be on the bridge. The cases of only partial moving load need not be considered at all, except for a few members where the moving load giving the maximum stress is only 933 ft. long. The moving load, according to its position on the bridge, produces changes in the length of the cables. It lengthens them when it is in the center; it shortens them, as compared with the mean length, when it is near the ends. Mr. Mayer.

The length of the cables must be ascertained for the mean temperature and the moving load 933 ft. long, extending from one end toward the center, and also for the mean temperature and the moving load 1 000 ft. long at the center of the span. When the cable has the mean length between these two it should have the shape of one parabola extending from end to end of the span. If this adjustment is made, then the direct stresses in the chords of the stiffening trusses due to the moving load near the center of the span are negative, while the stresses in the chords of the trusses due to the stretch of the cables produced by this position of the moving load are positive.

To obtain the actual maximum stresses, the two kinds must be subtracted. If, therefore, the stresses due to the stretching or shortening of the cables produced by the position of the moving load are neglected, the results obtained are too large. If the moving load is near the ends of the span, then the cables are shorter than the mean length. In this position the moving load produces positive, direct, stresses in the chords of the trusses, the shortening of the cables due to this position produces negative stresses in the chords, and the two must be subtracted to obtain the actual stresses.

For a two-hinged truss the opposite of these assertions is approximately true, at least for the chord stresses in the middle half of the span, and the total effect of the stretching of the cables must be taken into consideration. This difference was not considered by the other critic when he calculated the stresses due to this cause in three-hinged and two-hinged trusses. He states that the moment in a three-hinged truss due to the stretching of the cables is 40% of the greatest moment from the same cause in a two-hinged truss. His calculation is defective, as it is based on the assumption of a uniform moment of inertia. It neglects the fact that only a small fraction of this moment need be considered in the three-hinged truss, while the whole must be considered in the two-hinged truss. It also exaggerates the importance of these moments in a three-hinged truss, as it overlooks the fact that the economical depth of the three-hinged truss is at least twice that of the two-hinged truss for the same span.

It is stated in the paper that the assertions made are not necessarily true of bridges of very different span.

The writer's formulas for temperature stresses, with the corrections due to the elasticity of the trusses, will give those stresses with an

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Mr. Mayer. error of less than 3 per cent. They do not pretend to give the stresses which would occur if the dip of the cables were doubled. This, however, is hardly a serious drawback.

As stated in the paper, the stresses due to that change in length of the cables which is due to the position of the moving load should be subtracted from the moving-load stresses. This has not been done. It would be approximately correct to calculate only the stresses which are produced by the difference of the change in length of the cables which is produced by changes of temperature and that produced by changes in the position of the moving load.

Though the writer has under-estimated the deflection at the center of the span for three-hinged stiffening trusses, the stresses due to the change in length of the cables given by him are yet too large, as only those stresses are wanted which occur at the same time as the maximum moving-load stresses. These stresses are so insignificant in three-hinged trusses that the old practice of neglecting them is entirely justified.

This subject has been taken up, not because the writer thought the stresses important, but because their amount has been greatly exaggerated by others.

The stresses in the chords of three-hinged trusses, produced by changes in the length of the cables, vary with the change in the depth of the trusses in the same ratio as the stresses due to an unequally distributed moving load. The moments producing these two kinds of stresses decrease as the depth of the trusses decreases. If the moments for a given depth of truss are M_m and M_l, then the moments for another depth are $a\,M_m$ and $a\,M_l$.

If the two depths are d_1 and d_2, then the stresses of the two kinds are, for the two different depths, $\frac{M_m}{d_1}$ and $\frac{M_l}{d_1}$ for the depth d_1; and $a\frac{M_m}{d_2}$ and $a\,\frac{M_l}{d_2}$ for the depth d_2. The ratio of both kinds of stresses for the different depths is $\frac{d_2}{a\,d_1}$. With change of depth of truss the two kinds of stresses change, approximately in the same ratio, but not in the ratio of the depths. Each kind, therefore, remains the same proportion of the sum of the two. If the total unit stress is the same in both cases, the unit stress from each kind of stress must be the same. If the stiffening truss were perfectly rigid, a would be equal to 1, and both the temperature and moving-load stresses would change inversely as the depth of the truss. Both stresses are affected in the same manner by the elasticity of the truss. The writer has nowhere stated, as asserted by his critic, that the temperature stresses per square inch of chord section vary as the depth of the trusses. Every one of the writer's statements on this subject he believes to be true and proven by the argument.

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Mr Mayer.

The writer is misquoted by one of his critics when he states that $(w + q)$, in the formula for the calculation of the temperature stresses, means the dead load per linear foot of bridge. q is the suspender pull from dead and live load; w is the weight of the cables, both per linear foot of bridge.

The objection has been made that the heights chosen for the three-hinged and two-hinged trusses are arbitrary. The reasons governing the selection of the least height are very complicated, especially for two-hinged trusses. The greatest permissible grade prescribes a certain minimum height. The stresses due to change in the length of the cables increase with the height of two-hinged trusses, and prevent the adoption of as great a height as would be economical in their absence. In shallow trusses a diminution of the height diminishes considerably the moments and shears which must be resisted by the trusses. This fact is in favor of a small height. The maximum grade would increase with a diminution of the height adopted by the writer for two-hinged trusses and would become objectionable, if it is not so already, if a smaller height were chosen. The heights selected for two-hinged trusses are practically those which have been adopted in the two designs for the Hudson River Bridge which were published some years ago. A considerable change, within practicable limits, in the height chosen for the two-hinged trusses would have little influence on their weight.

In the half-spans of the three-hinged trusses, the distribution of the moments and shears is similar to that in ordinary trusses of the length of the half-spans, with the difference that all the moments and shears due to unequally distributed loads are positive and negative, and only about half as large in the stiffening trusses as in ordinary trusses; and that uniformly distributed loads produce practically no stresses in the stiffening trusses. The temperature stresses in three-hinged trusses are unimportant. The diminution of the moments and shears which must be resisted by the trusses, accompanying a diminution in the depth of the truss is of much less importance in the deep three-hinged trusses than in the shallow two-hinged trusses. The proper shape and depth of each half-span of a three-hinged truss is nearly the same as that of an ordinary truss as long as the half-span. The most economical depth is somewhat smaller than that of the ordinary truss, due to the differences previously stated. The subject is so complicated that the writer has thought it best to compare concrete cases, as a general comparison is practically impossible. The difference in economy between three-hinged and two-hinged trusses is so great that a small error in the selection of the proper depth will not modify the conclusions seriously.

The objection has been raised that three-hinged trusses require larger cables than two-hinged trusses. The opposite is true.

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Mr. Mayer. The maximum stress in the cables occurs when the bridge is fully loaded at minimum temperature. In this case the two-hinged truss is bent up by the cables. A slight increase of the suspender pull over the whole span is produced thereby, and the load on the cables is increased. With the three-hinged truss, a smaller increase of the suspender pull is produced, which is exactly balanced by a large decrease of the suspender pull at the center of the span.

Referring to the maximum grade at the ends of the span, one of the writer's critics calculates the grade, with two-hinged trusses of uniform moment of inertia, to be 0.8 of that with trusses of uniform chord stress per square inch. He arbitrarily assumes that the force producing the upward bending is uniform over the whole length of such a truss. This, evidently, would not be the case, but the force would be larger near the ends, where such a truss bends less than at the center. The value tan. $\alpha = 3.2\,\frac{\eta}{l}$, therefore, is smaller than the actual value for such a truss. The critic concedes that a well-proportioned truss would not have a constant moment of inertia, but that this would decrease in the end quarters. The actual condition, therefore, is intermediate between that assumed by the writer and that assumed by his critic. The angle α, therefore, will be intermediate between the two cases. The critic, on this account, makes a correction of 6%, but in the wrong direction; instead of increasing α, he decreases it. The correct value for tan. α, in the two-hinged truss, may be as small as $\frac{3.8\ \eta}{l}$, instead of $\frac{4\eta}{l}$, which was given by the writer. The critic then proceeds to compare his grossly under-estimated grade of the two-hinged truss with the grade of a shallow and flexible three-hinged truss, and finds, by means of formulas based on erroneous assumptions, that they are nearly alike. He overlooks the essential point of the whole controversy and the main purpose of the introduction of the center hinge, which is to make possible a greater depth of truss than is permissible in the absence of a center hinge. This deeper truss will deflect practically in two straight lines, as long as the load is uniform, and the correction he makes on account of the curvature of each half-span under uniform load over the whole span is grossly excessive. The true amount of this correction is so small that it may well be neglected. The curvature of the half-span becomes only important for the partial loading of the whole span.

The writer agrees with many of Mr. Hildenbrand's statements, referring to stiffening trusses of highway bridges, but these are beyond the scope of the paper. As there exists no long-span suspension bridge for railway trains, it is impossible to prove, by practical examples of the present, that the various stiffening systems, besides those here

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discussed, are inferior, and thereby to justify the neglect to consider them. The Grand Trunk Bridge at Niagara Falls was at one time considered fairly satisfactory. It was considered advisable, however, to run all the trains over it very slowly. At present, the writer believes that railway companies would insist on a much greater degree of rigidity than that bridge possessed, as they wish to run their trains over their bridges with much greater speed. The whole argument regarding the adequacy of a given stiffening system for a railway suspension bridge must be purely theoretical. As these stiffening systems are all extremely complicated in their nature, it is too much of a task, in this closing discussion, to undertake to prove their inadequacy or lack of economy; however, the writer thinks that the opinion against them is wide-spread, almost universal, and that the burden of proof lies with those who reject, and not with those who adopt it. Mr. Mayer.

The writer believes that there are great differences in the degree of possible accuracy in the calculation of different stiffening systems. Three-hinged stiffening trusses can be calculated without much difficulty, with errors of less than 3%; but stiffening trusses with two hinges, or none at all, cannot be calculated accurately; still less is this possible with the various mixed stiffening systems.

Mr. Hildenbrand asks: "What is a properly designed cable?" The writer's answer is: "One in which there are the smallest possible bending stresses, or in which the stress in each of the wires of the cable is always as nearly as possible the same." In a bridge with only one span suspended from the cables, and in which the rear cables are of considerable length, their position changes much, according to the amount of moving load on the bridge. In this case it becomes very important to fix the position of the points of the cables near the anchorages where the strands begin to spread, as otherwise the stresses in the upper and lower strands become very different for different positions of the cables.

Near the saddles supporting the cables, at the anchorages, and on the towers the cables will be bent and straightened with their fall, rise and lateral deflection, on account of changes of temperature, load, and wind pressure. This bending and straightening of the cables would produce very large inequality of stresses in the different wires if tight wrapping extended up to the saddles. The wires themselves are bent and straightened at the saddles, producing bending stresses in them. A large radius for the bottom and sides of the saddles, where this takes place, therefore, is desirable. If a wire of 0.2 in. diameter is bent in a radius of 10 ft. the stress in the fibers at the outside of the curve is 23 300 lbs. per square inch more than the average stress in the wire. This bending stress can easily be reduced to one-third if the radius of that part of the saddle over which the alternate bending and straightening takes place is made 30 ft. instead of 10 ft. When

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Mr. Mayer. the suspenders are put on the cables, the latter are bent at the suspender saddles. This bending would produce large inequality of stresses in the different wires, if the cables were tightly wrapped. Wherever the cables are bent and straightened during the existence of the bridge, provision must be made for this bending to take place with a large radius, and the cables must be made to bend in detail by the sliding of the wires upon each other without excessive friction. Near all the places where such bending takes place, a tight wrapping of the cables is therefore objectionable.

The writer is not aware that he ever recommended small unit stresses in stiffening trusses. Mr. Hildenbrand neglects to state the kind of steel to which the writer's unit stresses refer. This is about 50% stronger than ordinary medium steel. The stresses adopted in the paper correspond to stresses of about 15 000 lbs. per square inch from live load, and 18 000 lbs. per square inch from live load, wind and temperature stresses combined.

It is suggested that the change in the dip of the rear cables, due to the presence or absence of the moving load on the main span, causes such great change in the elevation of the main span at the center that it might be preferable to suspend the rear spans from the cables. In the bridge used as an example in the paper, the dip of the rear cables of the 1 740-ft. span varies from 28 to 22 ft., with changes of load and temperature. This change in dip of the rear cables of 1 740-ft. span causes a motion of 6 ins. of the saddles on the adjacent tower, and, with two-hinged trusses, a change in elevation of 9 ins. at the center of the main span. If both the rear spans of the cables were 1 740 ft. long, the total effect of the change in the dip of the rear cables at both ends would be a change in elevation of 18 ins. at the center of the main span. This increases the maximum grade by 0.21 per cent. With three-hinged trusses, the maximum grade, from the same cause, is increased by 0.16 per cent. The writer believes that this is not important enough, in a bridge with three-hinged trusses, to justify such a fundamental change in the design as suggested by the critic. Local conditions will determine whether the rear spans can with advantage be supported from the cables, and, generally, will have more weight in deciding this question.

Mr. Duryea has given a clear and popular statement of the main question under discussion, and this is recommended to those who are not prepared to give several months' study to the subject. He has made a valuable addition to the paper by estimating the difference in the cost of the bridge used as an example, caused by the difference in the design of the stiffening trusses.

The writer's purpose was to show a simple method of calculating all the stresses in the stiffening system of a suspension bridge with three-hinged stiffening trusses, and to give assistance in the selection

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of the type and the proper general dimensions of the stiffening system. Bridges of the type here discussed must necessarily be rare, as the conditions justifying their construction are as yet extremely exceptional. The conditions justifying such bridges will probably become more frequent in the near future, and it is hoped that the paper and its discussion will be of some use to those who will have to design them. Mr. Mayer.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 929.

STRESSES IN COLUMNS SUBJECT TO COMBINED AXIAL AND TRANSVERSE LOADING.

By CHARLES WORTHINGTON, M. Am. Soc. C. E.

The most common examples of columns of this class are those supporting elevated railroads, roofs of mill buildings, posts of portals, etc.

The common method, of assuming that the lower end of such columns be "fixed," and determining the bending stress in the columns by the elastic-curve formulas on this assumption, has never appeared to the writer as a satisfactory solution of the problem. In the first place, the condition of "fixity," as assumed in determining these formulas, is not present. If the foundation pedestal were of steel, and the column extended down into it sufficiently far, and without any play whatever, the column could then be assumed as fixed at its base in accord with the conditions assumed in these formulas, but the critical point would only be transferred from the bottom of the column to the bottom of the pedestal, and as most pedestals rest on material which is quite soft compared with their own, considerable deflection would occur before the pedestal reached a firm bearing. The anchor bolts, also, are assumed to increase the fixity of the column base, whereas they do not come into play until there has been sufficient deflection of the column to bring the base to a firm bearing against the nuts, and to stretch the bolts themselves, if they are of any great

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length. On account of the lack of rigidity at the bottom of the column and at the bottom of the pedestal, therefore, the bending of the column does not follow the lines given by the deflection formulas based on "fixity" of the end.

This assumption, moreover, fixes at once the size of the masonry pedestal, and leaves no chance to vary its size to suit local conditions, whereas, for example, in designing an elevated railroad through city streets, it is quite often necessary to limit the size of the pedestal, in order to clear conduits, etc., already in place, no matter how much additional material this may require in the superstructure. It is desirable, therefore, that the connection of the column to the pedestal be such that the size of each may be varied in order to strike a balance between the cost of the column and the cost of the pedestal, with a view to maximum economy for the structure as a whole.

Another factor which is neglected in the foregoing method is the effect of the direct axial load on the bending of the column. As soon as the column starts to bend under the horizontal load, the windward side of the base rises, thus tending to shift the point of bearing of the column on the pedestal to the opposite side, thereby producing a bending in the column in a direction counter to that produced by the horizontal load, and directly proportional to the amount of the axial load. With a view to establishing the exact relations between these forces and their effect on the column and the pedestal, the writer has prepared this paper.

The general type of column under consideration is illustrated in Fig. 1, which represents the outline of a column with a square base, anchored down to a pedestal of masonry, and supporting a vertical load, with horizontal forces applied, one at the top, and one in the opposite direction at a point below, representing the force produced by a knee-brace, or strut of some kind. For purposes of analysis, the different forces and reactions, with the resulting stresses produced in the column, will be determined separately, and then combined for the total net results.

Considering first the horizontal forces alone acting, Fig. 2 shows the effect on the column of a horizontal force, H, which is the total horizontal thrust coming on the top of the masonry, from wind, traction, etc., acting on the structure of which this column forms a part. The forces, at the top and at the point, X_1, of the amounts

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indicated, are those necessary to hold the column in equilibrium against this force, H, applied at the base, and the column will tend to bend as shown. The moment of the external forces about any point on the neutral axis of the column, distant x from the base (clockwise moments being considered positive and those in the opposite direction negative), will be

$$-Hx \quad \ldots\ldots (1)$$

and this moment will be a maximum at X_1, where its value will be

$$-Hb \quad \ldots\ldots (1a)$$

To the column thus bent, now consider the vertical load applied at the top. The column being lifted from the masonry at the windward side, the center of bearing on the masonry will be at the opposite corner, and this will produce in the column a moment in a direction opposite to that of the horizontal force, tending to counteract this force and tending to bend the column back at its lower end, as shown in Fig. 3. In fact, the deflection of the column from the horizontal force will be so slight that the distance, c_1, can be taken as equal to d, the half-width of the column, without any appreciable error. The moment of the external forces, about any point on the neutral axis of the column between the base and X_1, will be constant, and of the value

$$+Pd \quad \ldots\ldots (2)$$

Finally, the effect of the anchor bolts is shown in Fig. 4, where A represents an assumed force which the bolts on one side of the column will be finally designed to resist. The moment of the external forces at any point on the neutral axis of the column, between its base and X_1, will be constant, and of the value

$$+Ae \quad \ldots\ldots (3)$$

Summing up (1), (2) and (3), the total resultant moment, at any point on the neutral axis of the column between the base and X_1, and making $c_1 = d$, as above, will be (see Fig. 5)

$$M = -Hx + Pd + Ae \quad \ldots\ldots (4)$$

and the column section at this point must be designed to resist this moment, M, in combination with a uniformly distributed direct compressive stress, of the value P. In practice, columns of this class are generally made uniform in section throughout their length; in which case only the maximum moment, occurring at X_1, will be required, and this will be of the value

$$M_1 = -Hb + Pd + Ae \quad \ldots\ldots (4a)$$

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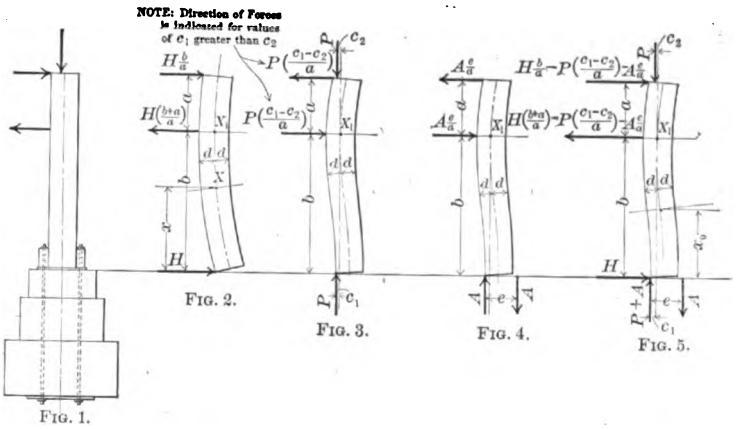

FIG. 1. FIG. 2. FIG. 3. FIG. 4. FIG. 5.

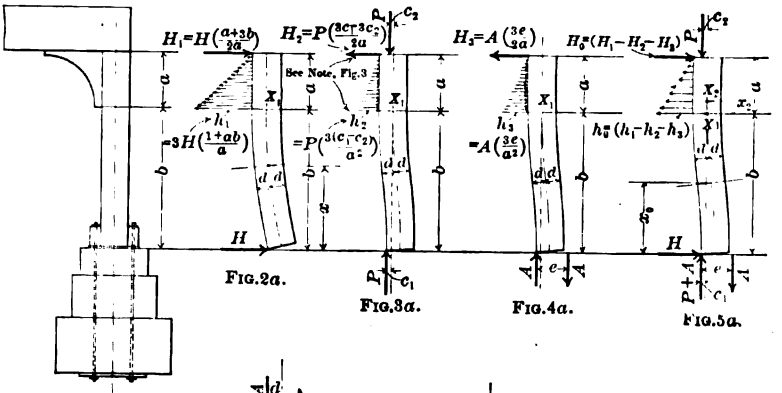

FIG. 1a. FIG. 2a. FIG. 3a. FIG. 4a. FIG. 5a.

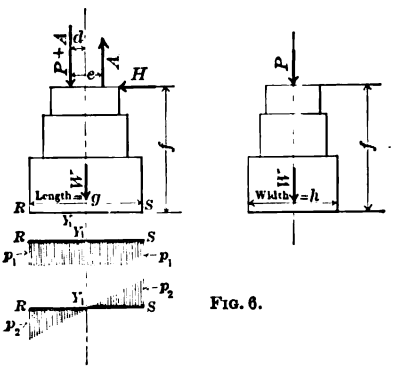

FIG. 6.

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The section of the column should be such that the total compressive stress on the extreme fibers at this point will not exceed a certain allowable unit value.

The "point of contraflexure" will occur where $M = 0$; from which the distance from the base of the column to this point will be

$$x_o = \frac{P\,d + A\,e}{H} \quad \ldots\ldots (5)$$

This is of no especial value, except for comparison with that method of designing columns of this class in which the location of the point of contraflexure is assumed at the start, and the column, anchorage, etc., is then designed to correspond.

The resultant force at the top of the column, for which suitable provision must be made, is, (assuming that $c_1 = d$, as before),

$$H\frac{b}{a} - P\left(\frac{d - c_2}{a}\right) - A\frac{e}{a} \quad \ldots\ldots (6)$$

If provision be made to ensure the load, P, being applied to the top of the column at its center, so that $c_2 = 0$, this force becomes

$$H\frac{b}{a} - P\frac{d}{a} - A\frac{e}{a} \quad \ldots\ldots (6a)$$

If the superstructure is connected to the top of the column so that the point of application of the load, P, shifts to one corner as the column bends under the application of H, then c_2 becomes equal to d, and this force becomes

$$H\frac{b}{a} - A\frac{e}{a} \quad \ldots\ldots (6b)$$

The forces given by Formulas (6), (6a) and (6b) represent also, in each case, the shear for which the web of the column and the rivets connecting it with the flanges must be designed, from the top down to the point X_1.

The force at X_1, for which provision must be made in connections, etc., is (assuming $c_1 = d$, as before),

$$H\left(\frac{b + a}{a}\right) - P\left(\frac{d - c_2}{a}\right) - A\frac{e}{a} \quad \ldots\ldots (7)$$

When $c_2 = 0$, corresponding with the conditions of Formula (6a), this force becomes

$$H\left(\frac{b + a}{a}\right) - P\frac{d}{a} - A\frac{e}{a} \quad \ldots\ldots (7a)$$

And when $c_2 = d$, corresponding with the conditions of Formula (6b), this force becomes

$$H\left(\frac{b + a}{a}\right) - A\frac{e}{a} \quad \ldots\ldots (7b)$$

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It will be noticed, by reference to the foregoing figures, that if the connection between the top of the column and the superstructure be such as to hold the top in a horizontal position under the application of H, the effect will be to produce a negative moment in the column above X_1, and to increase the horizontal forces at the top and at X_1, without, however, affecting the bending in the column from the point X_1 to the base. Care should be taken, therefore, in designing the details of this connection, to have the vertical load come upon the column centrally and remain so, as the column bends slightly under the horizontal load. This is often not feasible, and in these cases the forces for connections, etc., should be determined by Formulas (6*b*) for the top and (7*b*) for the point X_1, and then increased somewhat to provide for the effect of the column top being connected rigidly to the superstructure.

Below the point X_1, the web of the column and the rivets connecting it to the flanges must be designed for a horizontal shear, of amount

$$H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The base of the column should be designed so that it can receive, on one edge, without injury, a force of

$$P + A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and provision should be made for distributing this load on the masonry symmetrically about the corner of the column as the center of pressure, without exceeding the allowable limits of pressure on the masonry. With this in view, it is well to use a thick casting under the column proper.

The stresses in the column having been thus determined, it now becomes necessary to design the pedestal. The amount, direction, and point of application of all forces acting upon the pedestal are shown in Fig. 6, where P, A and H are the forces already determined, and W is the weight of the pedestal itself. It will be assumed that the pedestal rests upon a foundation of earth, on which the maximum permissible pressure per unit of area is p. Considering now the section $R\,S$ of the foundation, at the bottom of the pedestal, it must sustain a compressive load of $P + W$, uniformly distributed over its surface, and a moment taken about its center, of section Y_1, equal in amount to

$$M_f = Hf + Pd + Ae \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

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without exceeding the pressure, p, at the extreme leeward edge, R, and without tension at the opposite edge, S.

The base of the pedestal is assumed to be rectangular in section, of the dimensions shown, and symmetrical about the center of the column. Considering now the section $R\,S$, the reactions produced by the forces indicated are a uniformly distributed force, of value p_1 per unit of area, equal to

$$\frac{P + W}{g\ h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and a distributed force per unit of area, varying uniformly from a maximum of p_2, acting upward at the leeward edge, R, to an equal value of p_2 acting downward at the windward edge, S; the maximum value, p_2, of this force, per unit of area, being

$$M_f \left(\frac{6}{h\ g^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

To ensure a pressure on the foundation at the base of the pedestal not exceeding p on the outer edge, R, the value of $p_1 + p_2$ must be equal to or less than p; and to ensure there being no tension on the windward edge, S, the value of p_1 must be equal to or greater than p_2.

It will be noticed, by comparing Formulas (4) and (10), that increasing the force, A, acts against H, to diminish the total bending on the column, and with H, to increase the overturning on the pedestal. Increasing the value of d produces the same effect. In assuming the size of the anchor bolts and the width of the column, therefore, the relative cost of the column and the pedestal should be taken into consideration, with a view to maximum economy.

The value of P used should be the net value; for example, in a mill-building column it should be the total dead load coming on top of the column, plus the vertical reaction from wind for the leeward column, and minus that reaction for the windward column; in which case the total horizontal wind force coming on the building should be divided between the two columns, so that the combined stress from direct compression and bending will be the same for each column.

In Fig. 1*a* is shown in outline a modification of the general case under consideration. In this case a bracket takes the place of the knee-brace or strut of Fig. 1, and Figs. 2*a*, 3*a*, 4*a* and 5*a* show, as before, the induced forces necessary for equilibrium, corresponding to those shown for the first case in Figs. 2, 3, 4 and 5. In this case there

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is substituted, for the single horizontal force applied at X_1, a force extending over the distance a, and varying in value uniformly from 0 at the top of the column to a maximum of h_o at X_1.

The moment of the external forces, taken about any point on the neutral axis of the column, distant x above the point X_1, and making $c_1 = d$, as before, will be (see Fig. 5a)

$$M = -H(b+x) + Pd + Ae + \frac{h_0 x^2}{2}\left[\frac{x^2}{6}\left(h_0 + \frac{h_0 x}{a}\right)\right] \ldots\ldots\ldots (4b)$$

And this will be a maximum when $\frac{dM}{dx} = 0$; from which the value of x_2, the distance of the point of maximum bending above the point, X_1, is found to be

$$x_2 = \sqrt{\frac{4a^2}{9} + \frac{2Ha}{h_o}} - \frac{2}{3}a \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

and the moment at this point will be

$$M_2 = -H(b+x_2) + Pd + Ae + \frac{h_0 x_2^2}{2}\left[\frac{x_2^2}{6}\left(h_0 + \frac{h_0 x}{a}\right)\right] \ldots\ldots (4c)$$

The connection at the top of the column must provide for a horizontal force, of value H_0, and the bracket must be designed to withstand a force per unit of length varying uniformly from 0 at the top to h_0 at the distance a down from the top.

The observations on the effect of shifting the point of application of the load, P, on the top of the column, in the first case, apply similarly to the present case.

The point of contraflexure will be at the same location as before, and the masonry pedestal will also be the same as in the first case.

In the foregoing, the writer has not attempted to treat the subject under consideration exhaustively, but has confined his attention to the conditions which generally exist in the actual designing of columns of this class. No account has been taken of refinements which would not affect the final structure appreciably, and he has endeavored to deduce formulas which may be used as given herein by substituting the proper values of the forces and dimensions, when the loads are applied as indicated, or which, by a similar analysis, may be modified to suit the changes which may be necessary for any particular case.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 930.

THE PENNSYLVANIA AVENUE SUBWAY AND TUNNEL, PHILADELPHIA, PA.*

By GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER, Members, Am. Soc. C. E.

WITH DISCUSSION BY MESSRS. JOSEPH M. WILSON, CHARLES G. DARRACH, WILLIAM BARCLAY PARSONS, GEORGE A. SOPER, RICHARD L. HUMPHREY, GEORGE S. WEBSTER AND SAMUEL TOBIAS WAGNER.

HISTORY, AND SEWER CONSTRUCTION.

The early history, and the construction of the sewers, connected with the work of abolishing grade crossings on Pennsylvania Ave., in the City of Philadelphia, by what is known as the Pennsylvania Ave. Subway and Tunnel, has already been described by the writers.†

It is the purpose of this paper to take up the subject where the description was left off in the previous paper, and to present such features of the completion of the work as it is hoped may be of interest to the profession.

GENERAL DESCRIPTION OF THE WORK.

The following is a concise description of the work as a whole:

Beginning at the south end of the old bridge over Callowhill St., below Twelfth St., the grade begins to fall toward the west at the

* Presented at the meeting of May 7th, 1902.

† *Transactions*, Am. Soc. C. E., vol. xliv, page 1.

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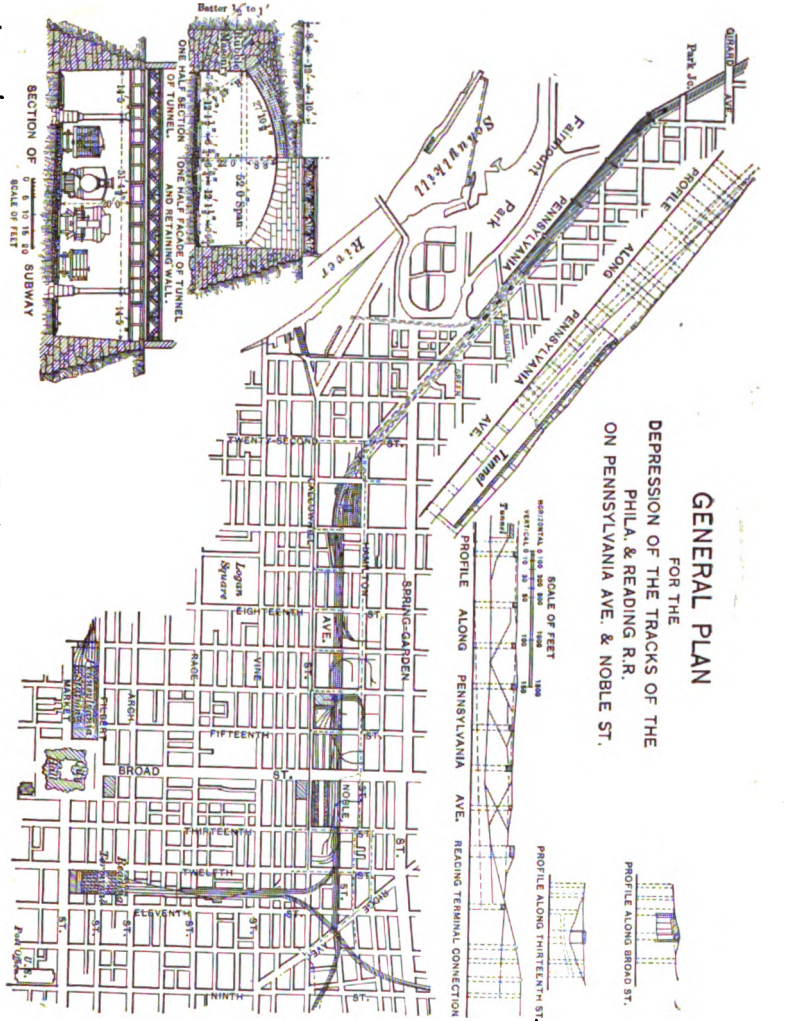

Fig. 1.

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rate of 1.46 ft. per hundred to Twelfth St. Between these points there is an iron viaduct which has been lowered from a higher elevation to meet the change in grade. A new, steel, half-through, plate-girder span, with two main girders, at 28½-ft. centers, and with a solid steel floor at right angles to them, crosses Twelfth St., and the grade of the street is slightly lowered so as to give 13 ft. headroom. From this point the grade descends at the rate of 2.5 ft. per hundred to the west, meeting the subway level west of the line of Broad St. See Fig. 1.

After crossing Twelfth St., the tracks, two in number, are supported upon earth filling between masonry retaining walls. At Thirteenth St. the most serious street change of grade occurs. On account of the rapidly descending grade of the tracks, the new level of the tracks would about cross the old level of Thirteenth St. at grade. In order to avoid this crossing it was necessary to depress Thirteenth St. about 13.5 ft. under the tracks, and run out to the present grade at Nectarine St. on the north and at Carlton St. on the south. This also changed the grades of Callowhill, Buttonwood and Hamilton Sts., all of which run east and west.

On the south side of Noble St., east of Thirteenth St., a coal yard of the Philadelphia and Reading Railway Company was lowered to the new grade of Thirteenth St., trestles were built in the yard, and track connections made from the tracks on Noble St. over the bridge at Thirteenth St. (See Contract 12.)

At Thirteenth St. the new freight yard of the railroad company commences. This extends from Callowhill St. to Noble St., and from Thirteenth St. nearly to Sixteenth St., and contains the new freight buildings (Contract 15), situated between Thirteenth and Broad Sts. At Broad St. the subway is about 25 ft. below the street level, and this depth, generally, continues to the west.

The Baldwin Locomotive Works, which are situated on the north side of the subway, between Broad and Fifteenth Sts., are connected with the depressed tracks by means of a side track on a 5% grade. On the south side a driveway, on a 5% grade, connects the subway yard with Callowhill St., and a second 5% grade connects the yard tracks with the high-level yard tracks in front of the warehouse at Sixteenth St.

On Broad St., the change of street grade at the north end of the bridge was about 4.5 ft. This was necessary in order to give a clear-

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ance of 18 ft. over the main tracks. The railroad clearances throughout, with this single exception, are 20 ft. over the main tracks and at least 15 ft. over all side and yard tracks. Broad St. is the first street to cross over the subway. All streets to the west, as far as and including Twenty-first St., are carried over by means of steel bridges of plate-girder, deck type. Between Twenty-first and Twenty-seventh Sts. the streets cross the tunnel; beyond Twenty-seventh St. the only means of crossing is by foot-way bridges, Fairmount Park, with no avenues, being on the south.

The original plans contemplated the erection of a large commercial coal conveyor for the retail coal trade, located on the south side at Fifteenth St. This was abandoned, and a small coal yard constructed in its place. The original structure was covered by Contract 18; the coal yard, by Contract 49.

At Sixteenth St., in addition to the highway bridge crossing the subway, a bridge on the street level is provided to cross from the railroad yard at Sixteenth and Hamilton Sts. to a large warehouse owned by the Philadelphia and Reading Railway Company, and also to provide connection to the coal yard at Fifteenth St. It is connected with the subway tracks by means of the 5% grade already referred to. At this point the subway narrows down to the bed of Pennsylvania Ave., 80 ft. between house lines. In some cases, between this point and Twentieth St., the retaining walls on either side are built under the fronts of the buildings, the wall being on private property. In others, the wall is placed about 15 ft. from the building line, and is entirely within the bed of the street. Provision is thus made for four, and in some cases six, tracks between the retaining walls.

Between Sixteenth and Seventeenth Sts. on the north is situated the works of William Sellers and Company, Inc.; on the south, at the time the work was started, was located the works of A. Whitney and Sons. The first provision for access to these properties was by means of a hydraulic lift (Contracts 20 and 40), located on the south side of the avenue, connection being made to the north by means of an overhead bridge. The lift has been abandoned, the railway company has purchased the Whitney property, and the connection to the works of William Sellers and Company has been made by means of a 5.5% grade on the south side and an overhead bridge into their works (Contract 44).

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The Baldwin Locomotive Works occupy the property on the north side of the avenue, between Seventeenth and Eighteenth Sts. Connection is made by a siding, on a 5% grade, leaving the level of the subway west of Eighteenth St. On the south side there is no railroad connection.

Between Eighteenth and Nineteenth Sts. there are no properties with railroad connections. A driveway, on steel construction, is provided for street purposes at the east end of the block on the north side.

The Twentieth St. Elevator of the Philadelphia Grain Elevator Company is situated on the south side between Nineteenth and Twentieth Sts., and the works of Stanley G. Flagg and Company on the north. The original plans provided for access to these plants by means of a hydraulic lift and a bridge similar to that at Seventeenth St. This, however, was abandoned; a 4% grade was made connecting with the grain elevator, and a connection to Flagg and Company was made from the subway level (Contract 41). In this latter case the owners have erected upon their own property a lift capable of raising a single car; the City and the railway company constructing the necessary retaining walls, making all excavation, reconstructing the buildings, and making the track connection.

The railway company owns the property between Pennsylvania Ave. and Hamilton St. and between Twentieth St. and the intersection of Hamilton St. and Pennsylvania Ave. This space was formerly occupied by a freight yard, and by engine-houses and a repair shop. It has been excavated to the level of the subway, and an engine-house, freight-house, and repair shop (Contract 25) has been built, also a locomotive coaling station (Contract 26), a 50-ton electric crane (Contract 27), and an engine turn-table. Twenty-first St. crosses this yard by means of a plate-girder, deck bridge.

On the south side of the yard is the plant of Messrs. Bement, Miles and Company, reached by connections made at the subway level between Twentieth and Twenty-first Sts. The Knickerbocker Ice Company's plant, on the south side, west of Twenty-first St., is reached by means of a 5% grade.

At the west end of this yard, the tunnel (Contract 33) begins, and extends for a distance of 2710.42 ft. It provides for four tracks. On emerging from the tunnel, the portal of the Baltimore and Ohio Tunnel

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is passed, some 263 ft. to the west, and, following the grade of the Baltimore and Ohio tracks, which is 1.3%, the present grade is reached at Thirtieth St., the end of the work, and 10 000 ft. from the starting point.

Alignments and Grades.

Beginning at Callowhill St., below Twelfth St., and at Station 1, the alignment is as follows:

Station 1. P. C. C. to left. Radius 395.37 ft., for 54° 59′ 50″.

Station 5 + 65.1 P. C. C. to left. Radius 536.68 ft., for 18° 29′ 10″.

Station 7 + 38.27 P. T. Tangent running west in Pennsylvania Ave.

Station 44 + 20.9 P. C. (5° curve to right). Radius 1143.42 ft., for 36° 16′ 56″.

Station 51 + 44.97 P. C. C. Radius 13189.127 ft., for 4° 44′ 26″.

Station 62 + 36.22 P. T.

Station 73 + 82.11 P. C. to left. Radius 5715.36 ft., for 3° 24′ 47″.

Station 77 + 22.58 P. T.

Station 97 + 56.66 P. C. to right. Radius 1270.39 ft., for 22° 0′.

Station 100 Connection with old tracks.

The grades are shown in Table No. 1.

TABLE No. 1.

Station.	Elevation.	Rate.	To Station.	Elevation.
0	57.33	0.00	0 + 96.92	57.33
0 + 96.92	57.33	− 1.46	5 + 44.71	50.81
5 + 44.71	50.81	− 2.5	16 + 89.21	22.20
16 + 89.21	22.20	− 0.924	23 + 16.65	16.40
23 + 16.65	16.40	0.0	24 + 91.55	16.40
24 + 91.55	16.40	+ 0.805	36 + 71.87	20.00
36 + 71.87	20.00	0.0	40 + 74.21	20.00
40 + 74.21	20.00	− 0.376	51 + 39.58	16.00
51 + 39.58	16.00	+ 0.078	76 + 98.50	18.02
76 + 98.50	18.02	+ 1.30	97 + 00	44.02
97 + 00	44.02	+ 0.833	100 + 00	46.52

Division into Contracts.

In the spring of 1896 it was decided to advertise for bids for as much of the work as possible, and it was therefore divided into contracts arranged so as to enable it to be attacked from the greatest number of points at one time without interference, and at the same time divide it so that work requiring special skill in its execution could be separated from the rest in order that the number of sub-con-

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tractors would be reduced to a minimum. The following is a list of the contracts into which the work was divided:*

Contracts 1 to 6.....Sewers.
Contract 7.....Test pits.
Contracts 8 and 10..Removing old buildings.
Contract 9..... ...Removing surplus material.
Contract 11.....Temporary tracks.
Contract 12.....Work of construction east of Thirteenth St.
Contract 13.....Retaining walls from Thirteenth to Sixteenth Sts., except on the north from Broad to Sixteenth Sts.
Contract 14..... Excavation of the core between the retaining walls, and the construction of bridges from Thirteenth St. to the east tunnel portal.
Contract 15.....Freight-houses between Broad and Thirteenth Sts.
Contracts 16, 17, 19, 21, 22, 23, 24 and 28, } Retaining walls, and the reconstruction of buildings from Broad St. to the east tunnel portal.
Contract 18.....Commercial coal conveyor, Fifteenth St.
Contract 20.....Hydraulic lift at Seventeenth St.
Contract 25.....Freight-house, engine-house and repair shop, Twentieth St. yard.
Contract 26.....Locomotive coaling station, Twentieth St.
Contract 27.....Electric crane, Twentieth St. yard.
Contract 29.....Subway and tunnel from the east tunnel portal to Thirtieth St.
Contract 30.....Permanent tracks.
Contract 31.....Interlocking signal plant.

At the time of the initial preparation of this list, Contracts 1 to 10, inclusive, had been executed, and most of them completed. Plans and specifications, therefore, were prepared for Contracts 11 to 31, inclusive, the work covered by them consisting of all the principal items except the sewerage system, and covering the new buildings, track, signal system, etc., required by the Philadelphia and Reading Railroad Company.

Detailed plans of the freight and engine-houses at Thirteenth and Twentieth Sts., Contracts 15 and 25, were prepared by the then Chief Engineer of the Railroad Company, Mr. H. K. Nichols. The plans for work on the commercial coal conveyor, hydraulic lift, locomotive coaling station, electric crane and interlocking signals, were not prepared in detail, but bids were asked upon general plans and specifications,

* A complete list of the contracts is shown in Table No. 16, appended to the paper.

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the bidder to submit his detailed plans and specifications for approval. All the remainder of the work was prepared by the Bureau of Surveys, Department of Public Works of the City of Philadelphia, and approved by the Consulting Engineer of the Railroad Company, Joseph M. Wilson, M. Am. Soc. C. E.

On April 8th, 1896, the work was advertised, and May 12th was set as the time for the receipt of bids. It was thus expected that an ample opportunity would be given to prepare intelligent bids on any part of the work. During the time that the drawings, which numbered 164, were on exhibition, twenty-one contractors received prints which were prepared for their accommodation.

On May 12th, 1896, eighty-seven separate proposals were received from twenty-one different bidders, and the bids were scheduled and reported to the Director of the Department of Public Works.

The sum of the lowest bids received on this date amounted to $3 817 961.84.

Studies for Railroad Connections.

The method of making connections to the various industrial establishments along the line of Pennsylvania Ave. was one of the most difficult problems in the design of the work. The usual block distance between cross-streets is 396 ft., and the elevation to be overcome is from 25 to 30 ft. This made it very difficult to make connections, with moderate grades leading to the subway, without interfering with the grades of the intersecting streets.

Among the methods considered was the use of the rack-rail system with grades of 10 and 12 per cent. Studies were prepared for the connections to the following points on the upper level, using this system, *viz.:*

Warehouse at Sixteenth St.

William Sellers and Company, and Whitney, between Sixteenth and Seventeenth Sts.

Baldwin Locomotive Works, between Seventeenth and Eighteenth Sts.

Baldwin Locomotive Works, between Broad and Fifteenth Sts.

Philadelphia Grain Elevator, and Flagg and Company, between Nineteenth and Twentieth Sts.

Knickerbocker Ice Company, west of Twenty-first St.

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During the preparation of the plans, expert opinions were obtained from representative locomotive builders, and from the Engineer of the Abt Rack-Rail System in America. It was considered that heavy traffic, such as there is on Pennsylvania Ave., could be handled to advantage on a 10% grade fitted with a rack-rail.

The locomotive could be built so that it could be used on level tracks and on easy grades by adhesion, but have supplemental cylinders and cog wheels engaging in the rack where the 10% grades occurred. On a 10% grade a locomotive would have sufficient power to haul its own weight up the grade, and, in addition, a load equal to its own weight. By the addition of supplemental cylinders and cog wheels it would be possible to increase its power fourfold and more. The type of switching engine in use by the Philadelphia and Reading Railway Company, weighing about 57 000 lbs., with slight modifications, could be made to haul about 100 tons of cars and lading up a straight 10% grade. The cost of fitting up a locomotive with the necessary machinery for operating on the rack-rail was said to be from $2 500 to $4 000, depending upon the details of the plan adopted.

The cost of fuel and stores would be slightly increased, but probably not more than from 15 to 20 per cent. The additional cost of repairs was estimated at from $500 to $1 000 per annum per locomotive for the machinery for operating in connection with the rack.

The cost of the rack-rail for a single connection (say 200 ft.) would be about $2 600, including all appurtenances, and its life, as far as known, is very great.

As far as safety is concerned, the rack-rail on a 10% grade would be safer than a 5% grade without the rack, as the rack affords a means of controlling the load independent of the brakes. The rack-rail would in no way interfere with running standard cars over the tracks.

On such a system curves are especially to be avoided, if possible, and as long a radius should be used as circumstances will permit. Rack-locomotives have been built to operate on a grade of 33% combined with a curve of 398 ft. radius.

One of the four-wheeled Reading shifters in use to-day could be remodeled so as to haul a load of three or four loaded cars up a 10% grade fitted with a rack-rail, assuming each car to weigh 30 gross tons.

The principal reasons advanced for the use of the rack-rail were as follows:

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1st. On account of the steep grades used, the length of the connection is very much reduced, thus largely decreasing the cost of the work of construction and also damages to adjoining property.

2d. At many points it would have made it possible to connect with existing tracks on private property at their original level, instead of requiring a change of grade, with consequential damages.

3d. The present locomotives, with comparatively little expense, could be altered to suit the rack-rail; and the first cost of the rack-rail itself is comparatively small.

4th. The use of the rack-rail, even under disadvantageous circumstances, was found to result in a very considerable saving throughout the entire line.

The studies thus prepared did not meet the approval of the Philadelphia and Reading Railway Company, nor did they obtain the approbation of a large industrial establishment which at first was in favor of the plan. It was therefore abandoned, and the connections were made with 5% adhesion grades, together with the power lifts which have been referred to, and which in their turn were also abandoned, leaving all connections with the 5% grades, or directly from the subway level.

Sale of the Philadelphia and Reading Railroad.

On September 23d, 1896, all the property of the Philadelphia and Reading Railroad Company was sold at auction, at Thirteenth and Callowhill Sts., the City of Philadelphia, among others, protesting.

All the liabilities of the Philadelphia and Reading Railroad Company, in so far as they related to the work on the subway, were assumed by the purchasers, the newly organized company taking the name of the Philadelphia and Reading Railway Company.

Temporary Tracks.

Besides the local freight business of the Philadelphia and Reading Railroad Company to the industrial establishments on Pennsylvania Ave., it was necessary to operate the freight yards at Broad St. and at Front and Willow Sts., on the Delaware River, reached by way of Pennsylvania Ave. and Noble St. In order to provide for a continuance of this business, and at the same time to construct the subway, it was necessary to lay temporary tracks on Hamilton St. from Tenth

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St. to Pennsylvania Ave. and Twenty-second St., and on Pennsylvania Ave. from Twenty-second St. to Thirtieth St. The railroad company formerly had four tracks on Pennsylvania Ave.

On August 10th, 1896, the work of constructing the tracks on Hamilton St. between Tenth and Twenty-first Sts. was begun, the tracks being laid with new material. The street paving was removed and stored for future use, but a large portion of it was of cobble, which was never relaid. The subgrade was excavated so as to bring the top of the rail about level with the curb. Broken stone or approved slag ballast to a depth of 6 ins. was placed beneath the ties, which were of white, rock or chestnut oak, or yellow pine of good quality. The two ties at each joint were first-class, 8½ ft. long, 7 ins. thick, and not less than 7 nor more than 14 ins. on the broadest face. The remainder could be second-class, in which the face was not less than 6 ins. Interlaced ties were used for all turn-outs and cross-overs. Fourteen ties were used to each 30-ft. rail.

All new rails were of the Philadelphia and Reading standard 79-lb. section. The chemical composition was specified as follows:

Carbon	0.55 to 0.60
Silicon	0.15 to 0.20
Manganese	1.10 to 1.30
Sulphur	0.069
Phosphorus	0.06

Rails having carbon below 0.53 or above 0.65 were rejected.

The drop-test required that one test be made from each heat, and that 90% of the specimens must stand without breaking under a weight of 2 000 lbs. with a 20-ft. drop, and with the rail supported 3 ft. between centers.

It was the intention, in specifying this quality of rail, to take it up upon the completion of the temporary tracks, and use it for sidings and yard tracks in the permanent work, if it passed the physical inspection after being in service.

Most of the industrial establishments backing on Hamilton St. were connected to this temporary track system. At Sixteenth St. a connection was made with the tracks in Callowhill St., terminating in temporary freight-houses at Thirteenth and Callowhill Sts., and

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arranged so as to occupy only half of the block between Broad and Thirteenth Sts.

West of Twenty-first St., Pennsylvania Ave. was widened on the south side as far west as Twenty-fifth St., to provide for the new alignment of the tunnel, and between Twenty-second and Twenty-fifth Sts. the old tracks were shifted to the south, so as to allow the north abutment of the tunnel to be constructed. After the north tunnel wall was built, the tracks were shifted to the north side of the wall, where they remained until traffic was running on the permanent tracks through the tunnel.

It was the intention to use as much of the old rail as was possible in the construction of the temporary tracks, and, therefore, prices were asked in the proposal for doing the various classes of work, as given in Table No. 2.

At street crossings and other places where the tracks were crossed by teams or foot traffic the tracks were planked with 3-in. yellow pine planking on 1-in. blocking over the ties, so as to form a passage-way.

TABLE No. 2.—PRICES. TEMPORARY TRACKS. CONTRACT 11.

Classification.	Price.
New material throughout	$2.05 per linear foot.
Second-hand rails, new ties	1.85 " " "
Old rails, all other material new	1.25 " " "
Old rails and ties, other material new	0.99 " " "
New frog and switch	150.00 each.
New single and double slip-switches	650.00 each.
New crossings—steam or trolley	250.00 each.
Old switches, new ties and ballast	75.00 each.
Old single or double slip-switches	250.00 each.
Old crossings	75.00 each.
Lumber for planking	30.00 per M. ft. B. M.
Temporary freight-sheds	9 700.00
Shifting trolley tracks	1.25 per linear foot.
Drainage, and removing old buildings	3 700.00

All track work was made and paid for by the linear foot, measurement being made once by passing through switches on the main track, and again by starting from the point of the switch and measuring out on the turn-out.

The contract was awarded to P. McManus, and the work was carried on as the progress of the work on the subway demanded. It was completed on June 12th, 1898. In all, 49 933 ft. of track of all

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grades were laid, 194 switches and crossings were placed in the line, and the total amount paid to the contractor was $144 028.99.

Retaining Walls and Underpinning.

Immediately after work was begun upon the temporary tracks, the several contractors for the underpinning of buildings and the construction of retaining walls between Thirteenth St. and the east portal of the tunnel were ordered to begin work. This work was divided into nine contracts, numbered as follows, 13, 16, 17, 19, 21, 22, 23, 24 and 28, and each contract generally embraced the work between two adjacent cross-streets, that is, from the west house line of one street to the west house line of the next, and on both sides of the avenue, and included the adjustment of the buildings and the construction of the retaining walls. Each contract also covered both bridge abutments on the street to the west.

The first operation was either to underpin the walls of the adjacent buildings or to reconstruct their fronts. The method used was shown on the contract drawings for each side of the avenue. These drawings usually were three in number, showing as follows:

1st. The conditions existing before any work was done;

2d. Any temporary work which was necessary;

3d. The work as it was to be completed.

Table No. 4 is a list of the buildings adjacent to the work, giving the method of treatment.

TABLE No. 3.

Between streets.	Side.	Name of building.	Treatment.
Broad to 15th.....	North.	Baldwin Locomotive Works.	None required.
15th to 16th.....	North.	Four-story machine shop.	Reconstruction.
" " "	South.	Six-story warehouse.	Underpinning.
16th to 17th.....	North.	Large machine shop.	Underpinning.
" " "	South.	Car-wheel foundry.	Underpinning.
17th to 18th.....	North.	Baldwin Locomotive Works.	Underpinning.
" " "	"	" " "	Reconstruction.
" " "	South.	No buildings.	.....
18th to 19th.....	North.	Spring works.	Reconstruction.
" " "	South.	Row of small dwellings.	Removed.
19th to 20th.....	North.	Foundry and shop.	None required.
" " "	South.	Grain elevator.	Alterations made by owners.
20th to 21st.....	South.	Machine shops and foundry.	Reconstruction.

Underpinning.

Fig. 2 shows a typical section of the retaining wall, with the underpinning of the building on the north side of the subway, between Sixteenth and Seventeenth Sts.

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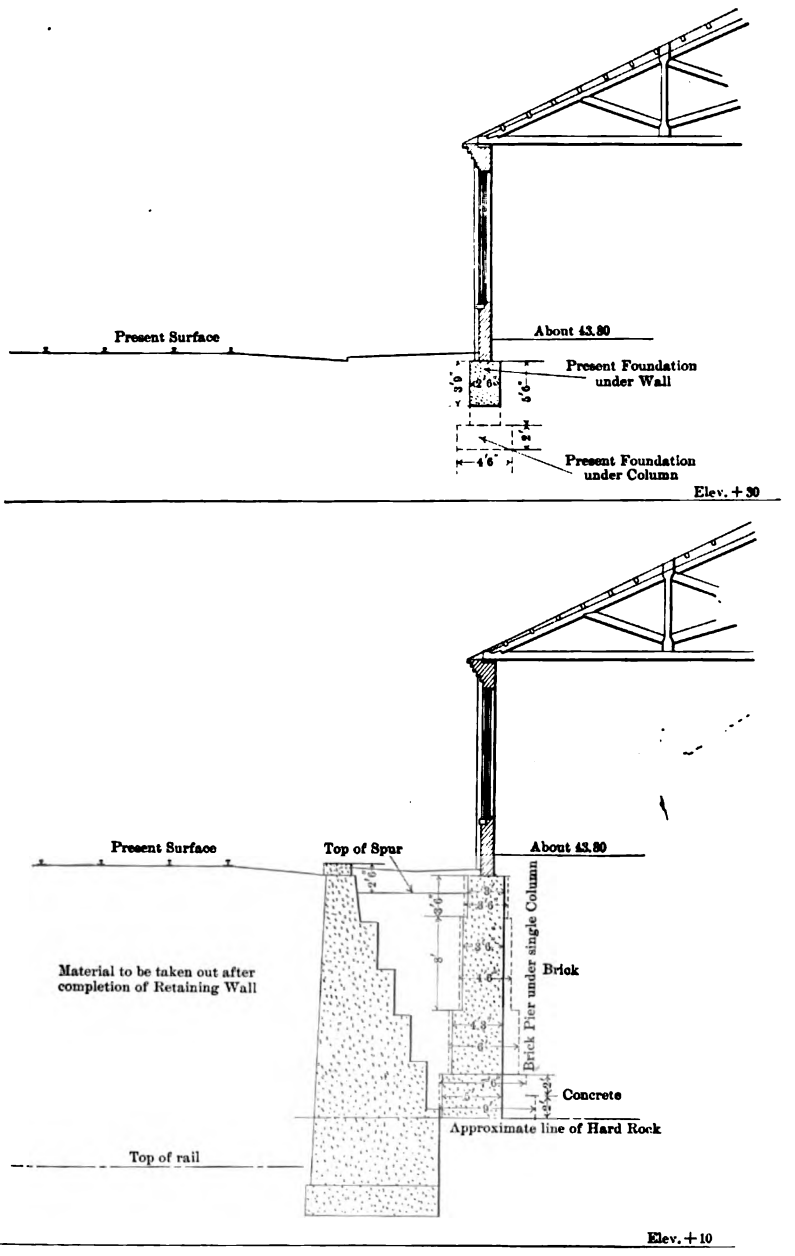

FIG. 2.

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Wherever it was very important not to enter upon private property, or where the conditions of the existing walls made underpinning cheaper than their removal, this method was always carried out. No specific method for doing this work was described in the specifications, the contractors merely being required to do it with the utmost care, and in the best manner, subject to the approval of the Chief Engineer; properly supporting the parts exposed through the excavation in such a manner as to prevent any damage to the building, machinery, or plant, either during the operation of the work or subsequent thereto. The detailed methods of shoring and making the excavations, therefore were left to the ingenuity of the contractor for each special case.

Fig. 1, Plate XIII, shows the method used for underpinning the steel columns in the front of the building of Messrs. William Sellers and Company, on the north side of Pennsylvania Ave. between Sixteenth and Seventeenth Sts. The columns supported the ends of the runway of a 50-ton traveling crane, spanning one of the shops and running at right angles to the avenue. The columns are in the front wall, which is of brick, 2 ft. thick and about 20 ft. high. The footing course was of rubble masonry laid in cement mortar and resting upon a bed of clay and gravel about 25 ft. thick, which rested on the rock. Small excavations were made under each column, and transverse wooden beams were slipped under its granite base and wedged up on a pair of longitudinal wooden stringers just outside of the footing. The excavation was then enlarged, and batter posts were placed in its sides. These carried the loads to sill pieces laid outside the limits of a trench which was carried down under the center of the wall. The excavation was sheathed and braced as shown in Fig. 3. A concrete footing was then placed upon the rock, and a vertical strut of 12 x 12-in. yellow pine was placed upon it, and wedged with steel wedges beneath the granite base. A brick pier, laid in Portland cement mortar, was then built up around this strut, and wedged up with a large number of steel wedges beneath the granite base. The intervening walls between the columns were treated in a simpler manner. Alternate sections, from 8 to 12 ft. in length, were excavated, shored and braced up, and then built up from the concrete foundation with selected flat stone laid in Portland cement mortar and wedged tight against the footing-course. The floor of this shop is a bed of concrete 12 ins. thick, and the large machine tools rest upon it without further foundations, many of them being immediately

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PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLVIII, No. 930.
WEBSTER AND WAGNER
ON PENNSYLVANIA AVENUE SUBWAY.

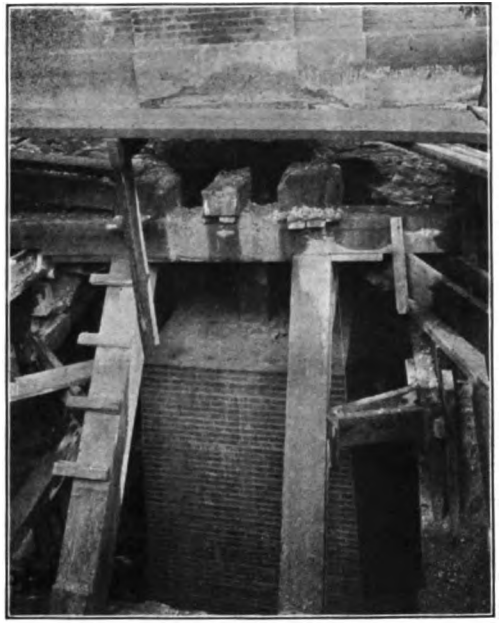

FIG. 1.—UNDERPINNING MASONRY.

FIG. 2.—EIGHTEENTH STREET BRIDGE.

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adjacent to the front wall. A very slight settlement of these tools would throw them out of adjustment, but no complaint of any settlement, whatever, was received, nor were any cracks visible in the front wall.

This method of underpinning was carried out successfully beneath the entire front of the six-story warehouse at Sixteenth St.; the eastern half of the Whitney Car Wheel Company, on the south side between Sixteenth and Seventeenth Sts.; the boiler-house and forge shop of the Baldwin Locomotive Works, between Seventeenth and Eighteenth Sts., and a large number of very heavy buildings on the line of Thirteenth St. In each case, the specifications provided for the general method

SYSTEM OF SHORING AS APPLIED FOR UNDERPINNING COLUMNS IN THE BUILDING OF WILLIAM SELLERS & CO.

NORTH SIDE OF PENNSYLVANIA AVE. BETWEEN 16TH AND 17TH STREETS.

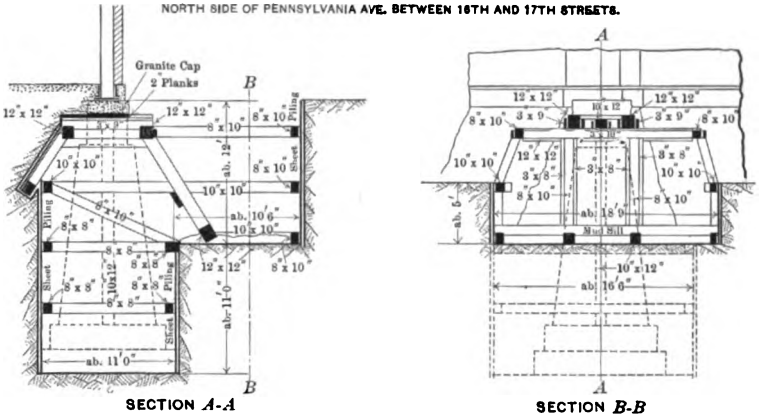

FIG. 3.

of the work, without regulating the details. In all cases, except for the work on Thirteenth St., the price for the underpinning was included in a lump sum for building the retaining walls and reconstructing the buildings in each block. On Thirteenth St., the price was by the cubic yard. A special price for additional underpinning, however, was bid under each contract, so as to provide for unforeseen conditions.

RECONSTRUCTION OF BUILDINGS.

Wherever it was not found expedient to underpin the buildings, the fronts were removed and reconstructed. This method was adopted:

1st. When the existing condition of the front wall was so bad as to make underpinning hazardous and expensive;

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2d. In cases where the main retaining wall had to be constructed with its face on the same line as the face of the building, the character of the wall requiring the use of derrick stone which could not be placed in underpinning;

3d. Where, for any reason, it was cheaper to remove the front than to underpin.

The specifications uniformly provided that a temporary front be constructed, inside the present front, made of hemlock scantling, covered inside with hemlock boards, Neponset roofing paper and second-quality white pine fencing vertically over all. The existing door and window frames were then to be removed from their places in the present wall, set in the temporary front, and the whole made weather-tight. The floor joists and other structural parts of the building were then supported as shown on the plans for each particular case, and the front wall was removed. Then the excavations were made for the retaining wall or the new foundation, and the new walls were constructed, the window and door frames being replaced as the building progressed.

The contractor on each contract bid a lump sum for removing all machinery, goods or stores which were in the way, and for restoring them to their original positions.

Fig. 4 shows a typical cross-section of such building reconstruction.

Retaining Walls.

The retaining walls were in all cases constructed of rubble masonry under the following general specifications:

"All retaining walls shall consist of first-class Conshohocken or other approved stone of good shape and good flat beds; no stone having less bed than face; laid on their broadest faces as rubble work in Portland cement mortar.

"In walls of 5 ft. thickness or less, the stones shall average from 6 to 8 cu. ft. each, and the length of the headers shall be two-thirds of the thickness of the wall. In walls over 5 ft. thick the stones shall average 12 cu. ft. each, and headers shall be at least 4 ft. long. Generally, no stones of less than 4 cu. ft. shall be used, except for filling the interstices between the larger stones. At least one-fifth of each face of all the walls shall be composed of headers placed so that those on one face come between those on the other; and, where the thickness of the walls is such that the headers from opposite faces do not overlap, stones not less than 4 ft. long shall be set transversely in the heart of the wall to complete the bond.

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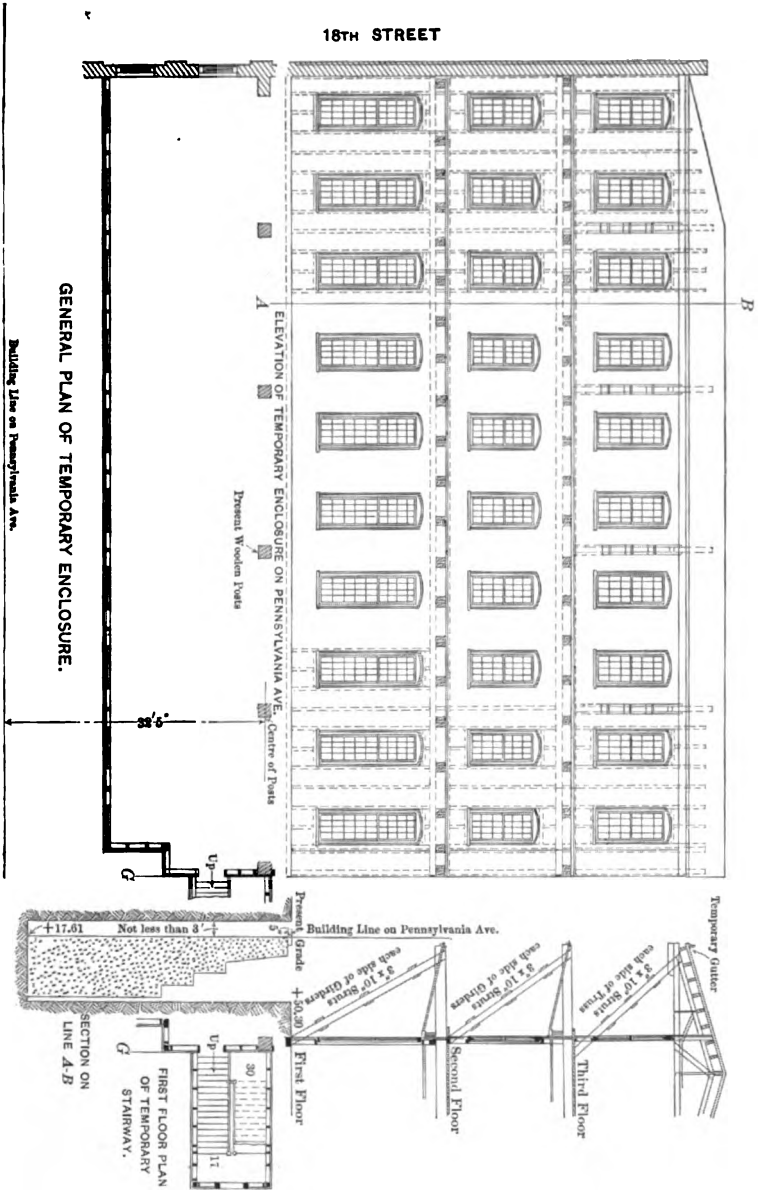

Fig. 4.

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"These walls must be built accurately to lines and levels given, and all exposed faces of the stone must be true and straight, and without prominent projections of the quarry faces. The mortar in all joints on exposed faces shall be kept back one inch.

"Selected stone shall be used at all angles, and shall be neatly pitched to true lines and laid on hammer-dressed beds. At all angles 1½-in. draft lines will be required.

"Grout shall be used in place of mortar, wherever directed by the Chief Engineer.

"The face and back of the wall shall be carried up together, over the whole wall, in approximately the same total height, except where otherwise specified.

"In rock-faced work, the faces of the stones shall have uniform projections not exceeding 3 ins., and in rough-pointed work not more than 1 in. beyond the neat lines, and in both cases they shall be pitched to a straight line at all the joints.

"The joints between all stones on the back of all retaining walls and abutments shall be carefully and thoroughly washed with Portland cement mortar so as to make the walls water-tight.

"In cases where rock adjoins the back of the wall, the wall is to be built up tight against the rock, and the joint thoroughly grouted with Portland cement grouting.

"All walls and abutments are to be thoroughly drained, wherever directed by the Chief Engineer, by means of 4-in. cast-iron pipe, built in the wall, or by openings left, as may be ordered. Wherever necessary, tile or French drains of broken stone, 18 × 18 ins., as shown on the drawings, shall be placed at the back of the wall. These drains and pipe will be paid for, per linear foot, at the price given in the proposal for such work.

"All pointing shall be done with Portland cement mortar of an approved brand. The surfaces of all stone shall be thoroughly cleaned, the joints scraped out to the depth of at least an inch, and the whole thoroughly wetted before pointing is commenced."

The mortar, grout and cement used were required to meet the following requirements:

"In all cases where Portland cement mortar is specified, the mortar shall be composed of 1 part of Portland cement to 3 parts of sand.

"Mortar taken from the mixing box, and moulded into briquettes, 1 sq. in. in cross-section, shall develop the following ultimate tensile strength:

AGE.	STRENGTH.
7 days (1 day in air, 6 days in water) 1 part of Portland cement to 3 parts of sand.....	100 lbs.
28 days (1 day in air, 27 days in water) 1 part of Portland cement to 3 parts of sand.....	150 "

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FIG. 1.—ERECTION OF BROAD STREET BRIDGE.

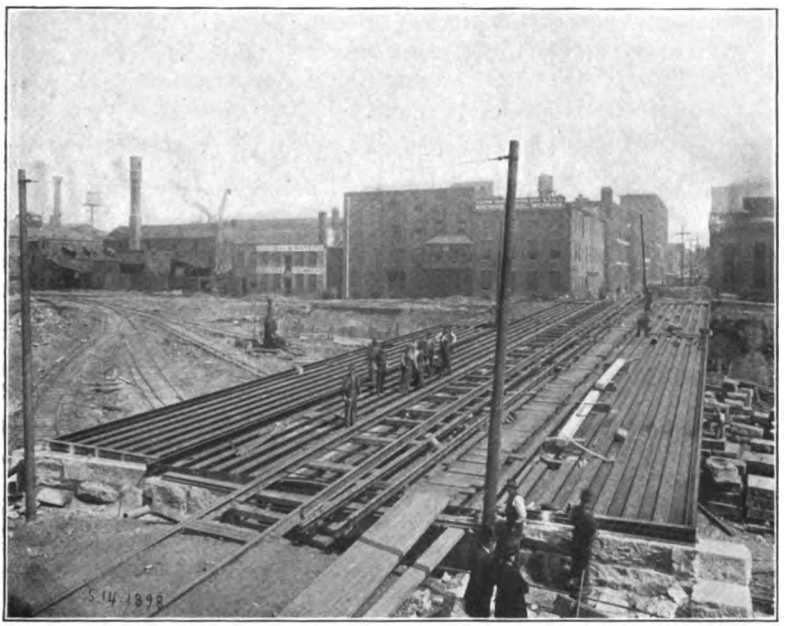

FIG. 2.—ERECTION OF FIFTEENTH STREET BRIDGE.

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"Grout made of Portland cement shall be composed of 1 part of Portland cement to 2 parts of sand, except where the foundations are wet, when the quantity of sand shall be diminished, making the proportions 1 part of Portland cement to 1½ parts of sand, which shall be used in the foundation masonry up to the neat lines if required.

"Portland cement shall have a specific gravity of not less than 3, and shall leave, by weight, a residue of not more than 1% on a No. 50 sieve, 10% on a No. 100 sieve, and 30% on a No. 200 sieve.

"Pats of neat cement, ¼ in. thick, with thin edges, immersed in water, after 'hard set,' shall show no signs of 'checking' or disintegration.

"It shall require at least 30 minutes to develop initial set.

"Briquettes of cement, 1 sq. in. in cross-section, shall develop the following ultimate tensile strengths:

AGE.	STRENGTH.
24 hours (in water, after hard set)	175 lbs.
7 days (1 day in air, 6 days in water)	450 "
28 days (1 day in air, 27 days in water)	550 "
7 days (1 day in air, 6 days in water), 1 part of cement to 3 parts of standard quartz sand	160 "
28 days (1 day in air, 27 days in water), 1 part of cement to 3 parts of standard quartz sand	220 "

"All cements shall meet such additional requirements as to 'hot water,' 'set,' and 'chemical' tests as the Chief Engineer shall determine. The requirements for 'set' may be modified where the conditions are such as to make it advisable."

Table No. 4 shows the average results of the tests of the cement used.

Fig. 5 shows a typical section of retaining wall with standard coping and French drain and weeper. Reduced heights were usually designed by cutting the standard section at the desired point.

Wherever possible, the faces of all walls were given a batter of ½ in. per foot. Where the face of the wall was on a building line, and in a few other cases, the batter was omitted.

The neat line was generally 18 ins. below the top of the rail. Unless otherwise ordered, the bed of the foundation was specified to be 5 ft. below the top of the rail in earth and soft rock, and 3 ft. below in hard rock. On lump-sum contracts, any masonry below these lines was paid for as additional masonry at the price bid.

Bridge Abutments.

Under the same contracts, and at the same time that the retaining walls were built, the abutments of all bridges were constructed. They were on the same lines as the walls, and were built practically in

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the same trench. The contract required the construction of temporary wooden bridges over the trenches on the lines of the cross-streets. The masonry was constructed beneath these bridges up to as high an elevation as possible, their completion being included in the contract for the construction of the bridges. These temporary bridges, therefore, were used over the two abutments of each cross-street until the core was removed and the bridges constructed. They were paid for at a unit price for the lumber, and generally cost about $250 each.

The abutments were constructed of rock-faced, coursed, ashlar masonry with rubble backing, and were laid in Portland cement mortar of the same proportions as described under "Retaining Walls." In all cases, a straight vertical joint was made between the retaining walls and the bridge abutments. The standard side and head clearances are shown in Fig. 6.

The specifications for the ashlar were as follows:

"In range ashlar the stones shall be equal to the best Stockton or Clearfield County stone. If other stones are intended to be used, specimen 6-in. cubes, properly labeled, must be submitted, and that accepted shall be considered as the standard. The stones shall be laid in regular courses with bonds on the face not less than 12 ins.; they shall be uniform in color, and free from stains. All mortar and grouting that has run out over the faces shall be washed off clean before it has set.

"If there be nothing shown or noted on the drawings, or stated in the clause relating to the work in particular of these specifications to the contrary, the faces of the headers in each course shall form at least one-fifth of its face.

"The height of the courses shall not exceed 30 ins., nor be less than 18 ins. The stones shall be dressed for ¼-in. joints, true to the proper lines, out of wind, with parallel beds and vertical joints; at the latter joints they shall be dressed at least 6 ins. back of the face, and the beds shall be tooled over their entire surfaces.

"The headers shall have at least as much width of face as rise, but not less than 18 ins., and they shall not be less than 4 ft. long.

"The stretchers shall have as much bed as rise, but not be less than 18 ins. wide; they shall not be less than 4 ft. long, and shall not break joints on headers.

"The backing shall be of coursed rubble masonry, well bonded with the ashlar and in itself."

All work on the bridge abutments was paid for by the cubic yard, *viz.*, excavation and masonry, including the rubble backing.

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TABLE No. 4.—Tests of Cement used on Pennsylvania Avenue Subway and Tunnel.*

Portland Cement.	Fineness. Percentage of Residue on Sieve.						Time of Setting, in Minutes.				Tensile Strength, in Pounds.																	
											Neat.										Three parts standard sand.							
	No. 50.		No. 100.		No. 200.		Initial set.		Hard set.		24 hours.		7 days.		28 days.		2 months.		3 months.		4 months.		7 days.		28 days.		2 months.		3 months.		4 months.	
	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.	No. of tests.	Average.
Vulcanite	58	0.5	58	7.0	49	20.1	60	124	61	350½	101	304	101	714	90	793	6	796	15	806	18	778	101	239	90	292	7	318	14	328	13	334
Giant	71	0.8	71	7.7	69	17.8	65	40½	66	132½	151	284	151	452	139	526	15	535	27	577	23	608	151	177	139	249	15	306	27	305	23	321
Atlas	15	0.5	15	9.2	15	25.1	36	68	36	339½	48	401	48	671	38	766	2	845	6	817	5	829	48	194	38	274	2	300	6	315	5	319
Alpha	12	0.9	12	9.5	12	23.8	4	190½	4	426½	24	381	24	675	24	759	4	808	7	828	5	830	24	247	24	289	4	326	7	318	5	344
Egypt	1	0.5	1	12.6	1	22.2	..	....	..	....	1	281	1	474	1	584	..	....	..	....	1	760	1	174	1	289	..	....	..	....	1	273
Heyn	2	0.4	2	7.2	2	17.8	8	100	8	246½	8	292	8	452	8	502	..	....	..	....	..	....	8	158	8	212	..	....	..	....	..	....
Saylor's	1	0.0	1	6.8	1	20.2	3	77½	3	435½	3	282	3	526	3	714	..	....	..	....	..	....	3	177	3	286	..	....	..	....	..	....
Totals and averages.	155	0.7	155	7.7	149	19.8	170	81½	178	268	336	314	336	579	308	656	27	656	55	698	47	704	336	208	308	267	28	311	54	314	47	32

Natural Cement.	Fineness. Percentage of Residue on Sieve.						Time of Setting, in Minutes.				Neat.										Two parts standard sand.							
Akron	..	....	..	....	..	....	7	81	7	252	7	186	7	271	7	396	..	....	..	....	..	....	7	134	7	223	..	....	..	....	..	....
Cumberland	7	3.8	7	14.4	5	28.2	18	123	18	66	18	129	18	253	18	323	1	360	2	362	..	....	18	142	18	244	1	223	2	371	..	....
Improved Union	4	2.4	4	10.0	4	20.4	20	4½	20	44½	21	166	21	219	16	299	..	....	..	....	1	358	21	150	16	222	..	....	..	....	1	335
Improved Anchor	1	3.5	1	18.7	1	29.1	2	28	2	148	2	114	2	209	2	392	..	....	..	....	..	....	2	128	2	217	..	....	..	....	..	....
Totals and averages.	12	3.3	12	13.3	10	25.2	47	82	47	54	48	153	48	239	43	330	1	360	2	362	1	358	48	143	43	231	1	223	2	371	1	335

* Compiled from reports of the Inspector of Cements, Bureau of Surveys.

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Cost of Underpinning, Reconstruction of Buildings and Retaining Walls.

A summary of the cost of the work under these headings is given in Table No. 5.

TABLE No. 5

Contract No.	Description.	Cost.
13.....	Retaining walls from 18th to 15th Sts., except on north side from Broad to 15th Sts., including both abutments at Broad St. bridge, south abutment of 15th St. bridge and excavation for lowering water mains on Broad St. E. D. Smith & Co., Contractors.....	$84 995.66
16.....	Retaining walls on north side from Broad to 15th St., including the north abutment of 15th St. bridge. M. McManus, Contractor.	24 295.21
17.....	Retaining walls between 15th and 16th Sts., including reconstruction of buildings on north side, underpinning of warehouse on south side, abutments of railroad high-level bridge and highway bridge on 16th St. P. H. Flynn & Co., Contractors.....	100 741.82
19.....	Retaining walls between 16th and 17th Sts., including underpinning of buildings on north and south sides, abutments of railroad bridge between 16th and 17th Sts. and highway bridge at 17th St. (This contract not completed on south side. See Contract 44 for balance of cost.) P. H. Flynn & Co., Contractors.....	108 018.49
44.....	Completion of masonry walls and abutment on south side between 16th and 17th Sts. E. D. Smith & Co., Contractors.....	13 444.48
21.....	Retaining walls between 17th and 18th Sts., including underpinning of buildings at east end on north side and reconstruction of building at west end on same side, construction of coal pit on north side and abutments of highway bridge on 18th St. P. H. Flynn & Co., Contractors.....	105 557.70
22.....	Retaining walls between 18th and 19th Sts., including reconstruction of buildings on north side and abutments of highway bridge on 19th St. P. H. Flynn & Co., Contractors.....	72 102.12
23.....	Retaining walls between 19th and 20th Sts., including abutments of highway bridge on 20th St. E. D. Smith & Co., Contractors.	68 812.68
41.....	Construction of pit for hydraulic lift on north side between 19th and 20th Sts., including bridge over connection, and reconstruction of buildings. E. D. Smith & Co., Contractors.....	19 904.20
24.....	Retaining wall on east side of 20th St. from Pennsylvania Ave. to Hamilton St.; on south side of Hamilton St. from 20th St. to east tunnel portal; reconstruction of buildings and construction of retaining walls on south side from 20th to 21st Sts.; both abutments of 21st St. bridge. P. H. Flynn & Co., Contractors.....	145 782.61
28.....	Retaining walls on the south side from the line of 21st St. to the east portal of the tunnel. E. D. Smith & Co., Contractors.....	16 284.20
52.....	Construction of coal-hopper pit and building on the south side between 20th and 21st Sts. Stacy B. Opdyke, Contractor.....	4 081.69
	Total work of reconstruction of buildings, underpinning and retaining walls.....	$763 970.81

The following is a list of the most interesting unit prices on the contracts mentioned in Table No. 5:

Ashlar for bridge abutments, per cubic yard.....$9 50
Additional rubble masonry, per cubic yard..... 6.50
Additional brick underpinning, per cubic yard.....$9.00 to 20.00
Additional rubble underpinning, per cubic yard.....$9.00 to 20.00
No. 4 railing, per linear foot..... 2.00
No. 6 railing, per linear foot.....$0.95 to 1.05
French drain, per linear foot..... 0.10
4-inch cast-iron weepers, per linear foot.....0 25 to 0.70

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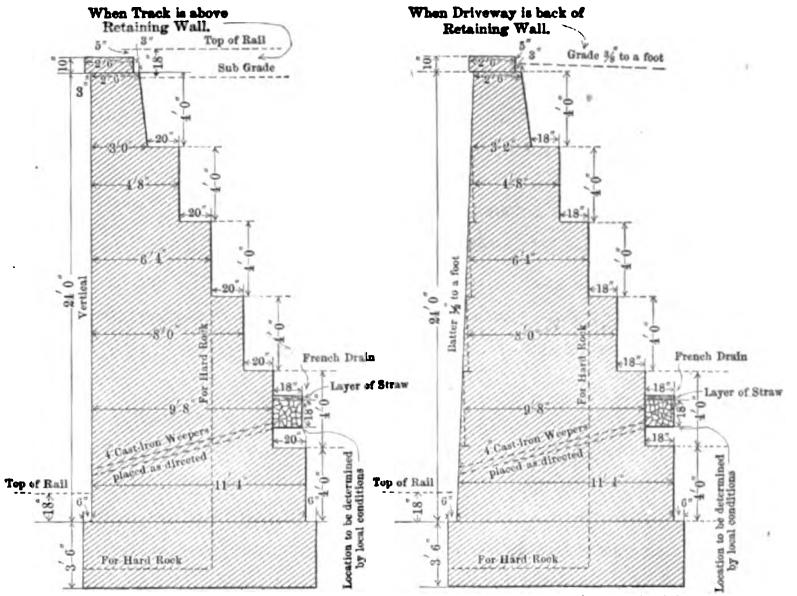
STANDARD SECTIONS FOR VERTICAL AND BATTERED RETAINING WALLS

PENNSYLVANIA AVENUE SUBWAY AND TUNNEL.



For the different heights of Retaining Walls, the Standard Sections will merely be cut off to the appropriate height measuring from the top downward, and the footing not less than 3 ft. 6-ins. high for ground and soft rock will have 6-in. projections front and back. From the line of hard rock down the back of retaining wall will be vertical and the footing, 2 ft. 6 ins. high, will have no offsets back; the Standard Section will be maintained above this rock line.

FIG. 5

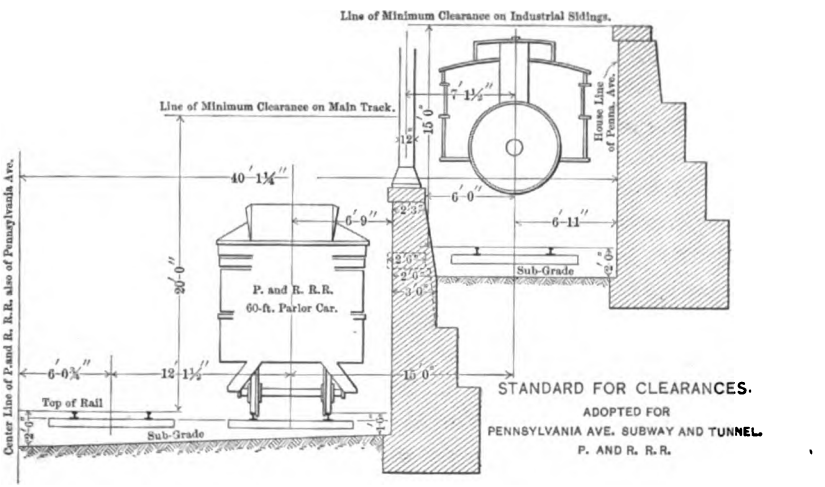


FIG. 6.

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In the above-mentioned total cost of the retaining walls, bridge abutments, and the underpinning and reconstruction of buildings from Thirteenth St. to the east portal of the tunnel, which amounted to $763 970.81, the piers for the bridges, a number of short lengths of retaining walls between the main walls, and in several cases the railings, were not included. The work which was thus omitted was all covered under Contract 14.

Proposed Tunnel Ventilation.

Contract 29, which covered the work of the construction of the tunnel, included a system of conduits designed for the artificial ventilation of the tunnel. This work was part of a complete system to meet the requirements of heavy traffic through the tunnel which was planned as 52 ft. wide and 2 913 ft. long.

Two fan stations (see Fig. 7) were proposed, situated so as to divide the length of the tunnel into four almost equal sections, the stations being located on the north side of the tunnel. In each station there were to be provided two fans, 20 ft. in diameter, drawing the foul air and gases from the tunnel through openings at intervals of about 150 ft., leading from the roof of the tunnel to the conduits which were also located on the north side of and parallel to the tunnel.

The grade of the conduits was arranged so that any condensation would run to the fan stations, where provision was made to care for it. The conduits were 11 ft. in diameter at the fan stations, decreasing to 6½ ft. at the ends. It was the intention to drive the fans by electric motors, and to discharge into ornamental stacks high enough to prevent any inconvenience to surrounding properties. The conduits at the stations were arranged so that the fans could be connected to either line of conduits, or so that one fan could exhaust from both lines of conduits in case of a break-down or repairs being made. Each fan was to have had a capacity of 600 000 cu. ft. per minute, or about one-fifth of the entire contents of the tunnel. It was thus expected that the entire contents would be replaced every five minutes.

On the south side of the tunnel, and placed midway between the suction openings, fresh-air intakes were to be provided, 150 ft. apart, each with an effective area of 35 sq. ft. They were to be connected to the tunnel at the level of the rail, and the openings into the street were to have been placed in the grass plats between the sidewalk and

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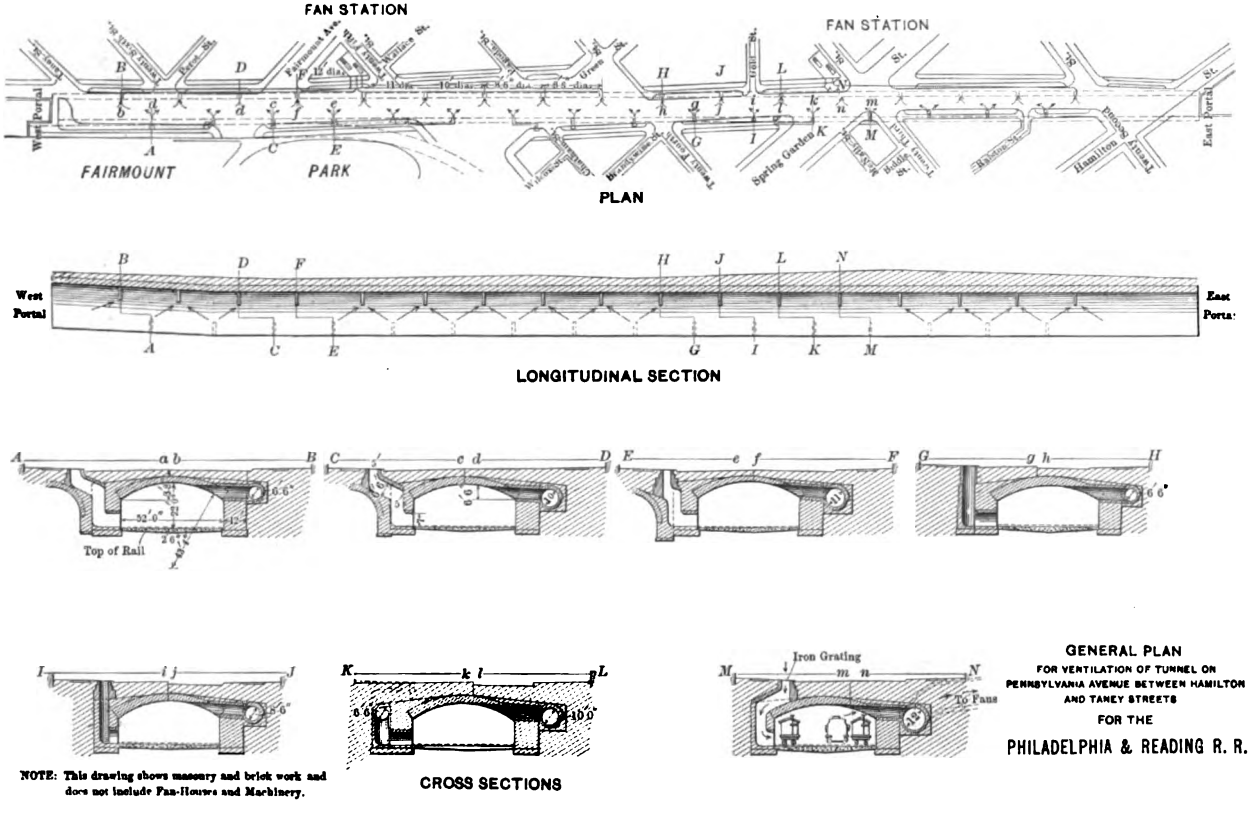

FIG. 7.

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the curb. If necessary, automatic shutters were to have been provided to prevent any gas from escaping into the street. These would close when there was any pressure of air from the tunnel. The power to operate the motors at the fans was to be obtained from a power-house situated on Pennsylvania Ave., between Eighteenth and Nineteenth Sts., which was also to supply power to the hydraulic lifts located at Seventeenth and Nineteenth Sts.

The Tunnel.

On May 28th, 1896, a conference was held at the office of the Mayor, Hon. Charles F. Warwick, between the officials of the City and of the Philadelphia and Reading Railroad Company, at which the City was asked to pay part of the cost of maintaining and operating the ventilation of the tunnel and the hydraulic lifts.

The agreement reached was to abandon the artificial system of ventilation in the tunnel, shorten its length, and introduce openings from the roof of the tunnel to the street.

These changes required a modification of the plans and specifications of the work on the tunnel. Work was at once begun on the plans, and, after several conferences, the revised plan was approved on August 3d by the Director of the Department of Public Works, and on August 6th by the President of the Philadelphia and Reading Railroad Company. Contract 29 was abandoned, and the work was re-advertised on August 15th, 1896, for proposals to be received on September 1st. Proposals were received on the latter date, on the new plans, under Contract 33.

Comparing the relative costs of the work, by figures based on the approximate quantities given in the specifications, there was a saving of about $220 000. Contract 29 did not include any work upon the fan stations for the ventilation system, nor the cost of the land for the same.

In the preparation of the plans for the new tunnel, it was necessary to make a large number of studies, as the interests of the City required as few openings as possible, and the Railroad Company desired practically an open cut. The conference above mentioned decided that each opening should be 10 ft. in width and of sufficient length to obtain 300 sq. ft. of net area. As the arch of the tunnel was designed of brick, it was decided that it would be impracticable to make the

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FIG. 1.—TUNNEL ARCH AND CENTERS.

FIG. 2.—INSIDE OF THE TUNNEL.

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construction of these openings of brick. The plans, therefore, which were approved, terminated the brick arch at each end of the ventilation opening, carried up the side-walls, and spanned the tunnel with steel girders and beams, between which the opening is framed.

On account of the action of the fumes from the locomotives upon the steel-work, it was considered of the first importance that every portion of the metal-work in these openings should be thoroughly protected. The following extracts from the specifications indicate clearly how this was done:

"These openings are to be constructed of a steel frame replacing the arch of the tunnel. Generally, four lines of plate girders, resting upon granite bridge-seats in pockets upon the side-walls of the tunnel, support lines of 12-in. I-beams, between which brick arches are to be constructed carrying the weight of the street above. The opening, 78 ft. long, has six lines of plate girders. The beams to be thoroughly tied together with tie-rods, as shown on the drawings.

"The spaces between the girders facing the openings shall be built up with a 9-in. brick wall, which shall have certain arches for manholes, as shown. These manholes shall be compactly built up, upon the completion of the work.

"The necessary retaining walls, upon the arch of the tunnel and surrounding the openings, shall be constructed as shown. The size of stone in these walls shall be subject to the approval of the Chief Engineer. The walls above the street level shall have a rustic combing of stone, carefully set.

"The metal-work shall be protected from the action of the locomotive fumes by plastering the main girders, where they cross the openings, with two coats of Portland or other approved cement mortar (mixed in the proportion of 1 part of cement to 1 part of sand). The first coat shall be about 1 in. thick, and, before the second is applied, the girder shall be covered with metal, plastering lath, or wire, of a quality to be approved by the Chief Engineer. The total thickness of the plastering shall be 2 ins. when finished.

"The under side of the main girders, not beneath the ventilation opening proper, shall be protected by a covering of 4-in. I-beams attached to the bottom flanges of the main girders, and supporting in turn a ceiling of terra cotta fire-proofing material arranged so as to cover the bottom flanges of the 4-in. I-beams in the manner shown on the drawings. The detail of the terra cotta to be subject to the approval of the Chief Engineer.

"This terra cotta ceiling to be set in Portland cement mortar (1 part of cement to 2 parts of sand) and filled up to a level on top with concrete composed of 1 part of Portland cement, 3 parts of

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sand and 5 parts of furnace slag, free from dust or dirt, and broken so as to pass through a 1-in. ring in any direction. The under side of the terra cotta material to be plastered with Portland cement mortar (1 part of cement to 2 parts of sand) 1 in. thick.

"The terra cotta work of the protective ceiling, including the concrete and the cement plastering, in place complete, as specified above, shall be paid for by the square foot at the price given in the proposal for the terra cotta protective ceiling or plastering of the main girders."

Tunnel and Open Subway at West End.

On January 12th, 1897, the contract for constructing the tunnel and the open subway at the western end of the work was awarded to Messrs. E. D. Smith and Company, and work was begun by them on March 8th, 1897. This contract was known as No. 33.

At this time the temporary tracks were constructed upon the south side of Pennsylvania Ave. from Hamilton to Twenty-fifth Sts., where they connected with the regular alignment. This allowed of the construction of the north wall of the tunnel from Hamilton St. to a point west of Twenty-sixth St., as well as the north retaining wall of the open subway from the last-named point to the end of the work at Thirtieth St. These walls were constructed in an open trench. The abutment wall of the tunnel was built to the upper end of the skew-back for the tunnel arch. The retaining wall for the open cut was completed and the coping and railing set. The temporary tracks were then shifted to the north of the walls, and they remained in this position until travel was begun on the permanent tracks in the subway.

On account of the close proximity of the tracks to the trench for these walls, it was necessary to construct, for a considerable distance, a trestle supporting the south track. This work was done under a separate contract, No. 42, which was awarded to Messrs. E. D. Smith and Company on August 24th, 1897. On account of some necessary delay in starting on this contract, the contractors for the tunnel, at considerable expense to themselves, began the work of constructing the south wall of the tunnel before the temporary track was moved to the north. This wall was constructed in the same way as the north wall, except where it immediately adjoined the wall of the Baltimore and Ohio Tunnel near Twenty-fifth St., at which point the walls of the two tunnels were back to back. In order to avoid any possibility of an accident, the subway wall was built in sections of 50 ft. each,

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the trench not being excavated in the alternate sections until the masonry on either side was well advanced. The back wall of the Baltimore and Ohio Tunnel was found to be in a very poor condition, and, as the subway wall was laid tight against it, each course was grouted in such a manner on the back that all the openings in the Baltimore and Ohio wall were filled. For this purpose, 322 batches of grout were used.

Steam shovels were then started at Thirtieth St., and a temporary incline was excavated to the west portal of the tunnel. The full section was then removed, approximately to the subgrade of the tracks, and centers were constructed for supporting the arch of the tunnel. These centers were designed by the contractors, and are shown in Fig. 1, Plate XV. They were built 50 ft. in length, and so as to run on 12 wheels on tracks placed adjacent to the tunnel wall on either side, the weight of a section being about 510 tons. An opening 23 ft. wide was left through them to allow of the passage of trains removing the excavated material. The ribs were 6 ft. from center to center, and were lagged with 3-in. plank. The brick in the tunnel arch and the stone for the spandrel filling were generally brought in on construction tracks on the upper level. Seven sets of centers were built and used for the arch, the centers remaining in place 7 days after the arch was keyed. After some experience, it was customary to move a center forward in about 20 minutes after it had been lowered to the tracks. As soon as the brickwork for a section of arch was in place, the spandrel filling, of stone laid in Portland cement mortar, was built, and the top plastered with natural cement mortar, $\frac{3}{4}$ in. thick. After this plaster had set, the work of back-filling to the subgrade of the pavement on the avenue was begun.

This work was carried forward from west to east. The brickwork was begun on May 30th, 1898, and was completed with the setting of the keystone of the east portal on December 17th, 1898.

After the excavation for the tunnel was completed, the shovels were removed again to Thirteenth St., and the old retaining wall on the north side of the cut of the Baltimore and Ohio Railroad was removed, and the excavation, as far as the west portal of the tunnel, was completed to subgrade.

The west end of the Baltimore and Ohio Tunnel, for a distance of 250 ft. from the west end, is covered with a steel roof. Where the

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back of the retaining wall supporting this roof, between the portals of the two tunnels, was exposed, it was faced with Leiperville stone and coped.

The work of placing the steel in the ventilation openings, of finishing around their tops, of placing the curb and paving the remodeled avenue, and of sodding and planting the trees and shrubs, followed in the order given. The final payment was made on June 5th, 1900. The completion of the work was delayed on account of some difficulty in fixing the paving grades at Fairmount Ave., caused by necessary conferences between the officials of the Union Traction Company and the Commissioners of Fairmount Park, which postponed the work of paving so late in the fall of 1899 that it could not be finished until the next spring.

Table No. 6 gives the average results of the tests of steel used. Four wooden, Howe-truss, footway bridges cross the four tracks of the subway and the two tracks of the Baltimore and Ohio Railroad on the lines of Thirtieth, Twenty-ninth, Twenty-eighth and Twenty-seventh Sts.

TABLE No. 6.—AVERAGE PHYSICAL AND CHEMICAL TESTS OF STEEL. (BASIC OPEN-HEARTH.) (CONTRACT 33.)

	No. of tests.	Elastic limit.	Ultimate strength.	Elongation in 8 ins.	Reduction of area.	Carbon.	Phosphorus.	Manganese.	Sulphur.
Specification requirements..	..	0.54 Ult.	57 000 to 65 000	25.00	...	...	0.04	..	...
Ventilation openings.... ...	92	37 880	60 270	29.90	57.5	0.166	0.030	0.42	0.038

The tunnel before described is 2 710.42 ft. in length between portals. It is 52 ft. in width between the side-walls, which are of heavy rubble masonry laid in Portland cement mortar, the face stone being a gneiss from Leiperville, Delaware County, Pa. For a distance of 45 ft. from the east portal the span is increased to 58 ft., so as to allow of the necessary side clearance to a turn-out leading to the railroad yard at Twentieth St.

The arch of the 52-ft. span is the segment of a circle of 43 ft. 8 ins.

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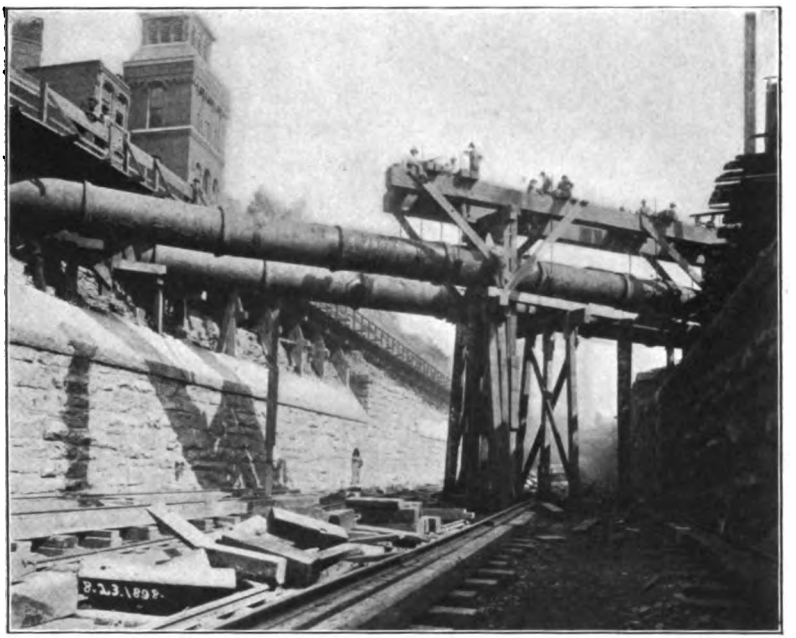

FIG. 1.—FORTY-EIGHT INCH MAINS, AT TWENTY-FOURTH STREET.

FIG. 2.—FORTY-EIGHT INCH MAINS AT TWENTY-FOURTH STREET.

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radius, with a rise of 8 ft. 8 ins., giving a clearance of 22 ft. over the rails at the center, with 18 ft. over the side tracks. It is composed of carefully selected, well-burned, hard, red brick, laid in natural cement mortar, and is 2 ft. 10½ ins. thick at the center and 3 ft. 11½ ins. at the skewback. The bonding is rather unusual, and is shown in Fig. 8. There are 7.25 cu. yds. of brickwork per linear foot of arch. The arch is not continuous, on account of the ventilation openings, of which there are thirteen. Twelve of these cut sections of 47 ft. 7 ins. in length out of the arch, and one of them occupies 78 ft. The construction of these openings has been described in detail. They are shown in Fig. 9.

The skewbacks are generally of Conshohocken stone of the dimensions shown in Fig. 8. Their cost is included in the price for the rubble masonry in the tunnel abutments above the neat line. These side-walls contain refuge bays, 5 ft. wide, 2 ft. 6 ins. deep, and 7 ft. high, arched with brick, and placed 50 ft. apart, alternately on either side of the tunnel. Upon the completion of the entire inside work of the tunnel, the masonry was thoroughly scraped, broomed and covered with Amphibolin waterproof whitewash.

All the water and gas mains and electrical conduits crossing the line of the tunnel were supported temporarily during the construction of the work. Nearly all of them were covered up in their original positions. The following exceptions occurred where there was not sufficient cover over the top of the brick arch to allow the mains to cross.

At Twenty-fourth St. two lines of 48-in. mains cross the line of the tunnel. On the line of Green St., and again at Fairmount Ave., single lines cross. At all these points the same method was used by Mr. Allen J. Fuller, then Engineer of Distribution, Bureau of Water, and now General Superintendent.

Special reducer pipes were manufactured. They are 37 ft. 7¼ ins. in length, horizontal on top and with the bottom 18 ins. higher at the center than at the ends. This makes a throat of 30 ins. diameter, the ratio between the throat and the main being as 1 to 2.51. The mains at Twenty-fourth St. are shown in Figs. 1 and 2, Plate XVI. The results of Mr. Fuller's experiments on the difference in head due to the velocity in the throat, as measured with a mercury column, are given in Table No. 7.

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TABLE No. 7.

Number of test.	Throat velocity, in feet per second.	Discharge in 24 hours, in millions of gallons.	Loss of head, in pounds.
1	4.504	14.3	0.018
2	6.777	21.5	0.044
3	10.111	32.0	0.100
4	11.287	35.7	0.133
5	13.450	43.6	0.188
6	14.358	45.5	0.184
7	14.767	46.8	0.213
8	16.381	51.9	0.286

Mr. Fuller, in commenting on these experiments, remarks:

"From the above given results it would appear that, in this case, the loss of head due to the reduced bore of the pipe is so slight that there can be little objection to the adoption of this method for crossing obstructions in the line of water or gas pipes or sewers. Care must be observed, however, in proportioning the throat or reduced area to the maximum required capacity of the conduit, otherwise the arrangement will prove unsatisfactory when discharging at a high velocity."

During the construction of the line at Twenty-fourth St., while the pipes were temporarily supported, a shifting engine and tender, on the temporary track adjacent, jumped the track, the tender falling 6 ft. and lodging upon the pair of pipes, which are about 14 ft. from center to center, and remaining there for about three hours, wholly supported by them. The accident did not break the pipes, nor cause any considerable leakage, although both mains were in service under a head of about 27 lbs. At the time of the accident the supports for the pipe in the excavation were as shown in Fig. 1, Plate XVI, the tender falling from the tracks on the left side of the photograph where the west pipe was unsupported for nearly four lengths.

The drainage of the depressed tracks is effected by means of a conduit of concrete and brick placed in the center of the tracks. It is connected at three points with the deep sewer on Pennsylvania Ave. to the north of the north retaining wall, and was constructed in 1894 and 1895. Two of these points are west of the tunnel; the third is beneath the tunnel on the line of Twenty-fourth St. The detail of this conduit is the same as that described in the open subway.

Pennsylvania Ave., over the tunnel, was widened from 80 to 120 ft. from Hamilton St. to Green St., there being two sidewalks, each 14 ft. in width, and two driveways of 27 ft. each, between the main curb and the curb of the ventilation openings. Sodded grass

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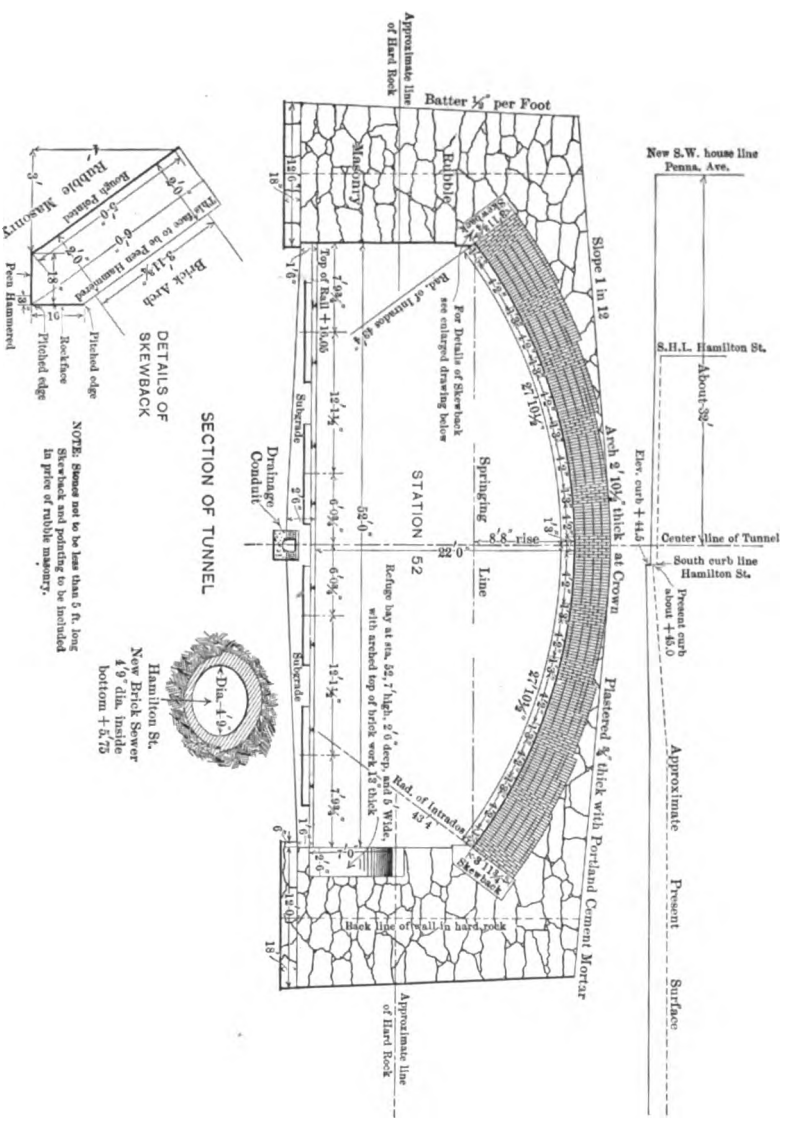

FIG. 8.

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plats, 6 ft. in width, are placed next to the main curbs, and are planted with Asiatic Buttonwood (*Platinus Orientalis*). The spaces around the ventilation openings are sodded and planted with shrubs specially selected to live under the conditions which exist. *Ampelopsis* vines are planted against the walls. The sidewalks are paved with brick, and the driveways with Trinidad Lake asphalt upon a cement concrete base, 8 ins. in thickness, the asphalt wearing surface, 2 ins. in thickness, being placed upon a 1-in. binder coat placed upon the cement concrete. All the curbing is of granite, 8 ins. in width and 24 ins. deep.

Table No. 8 gives the prices of the principal items which were used on this work. It should be noted that the excavation was measured without classification, and included all back-filling of retaining walls and abutments, over the tunnel and at other places, as directed. It also included the removal of all old walls and sewers; the maintenance of all travel; of water pipes, gas pipes, and electric conduits, and all sheeting and shoring where required.

TABLE No. 8.—Prices. Tunnel and Subway. Contract 33.

Item.	Price.
Excavation	$0.65 per cubic yard.
Rubble masonry, below neat line	5.00 " "
Rubble masonry, in retaining walls, above neat line	5.75 " "
Rubble masonry in tunnel abutments, above neat line, including skewbacks of tunnel arch	6.40 " "
Cut stone voussoirs in tunnel portals	18.00 " "
Granite bridge-seats, ventilation openings	33.00 " "
Brick masonry in tunnel arch	7.875 " "
Coping for retaining walls	1.50 per linear foot.
Asphalt paving, 8-in. concrete base	2.85 per square yard.
Brick sidewalk paving	0.90 " "
8-in. straight granite curb	1.40 per linear foot.
8-in. curved granite curb	1.60 " "
15-in. drainage conduit, in rock	2.00 " "
15-in. " " in earth	1.90 " "
12-in. " " in rock	1.90 " "
Grass sodding	0.60 per square yard.
Terra cotta covering, in ventilation openings, including cement, concrete, and plaster	0.44 " " foot.
Steel in ventilation openings	0.025 per pound.

The total cost of the work under this contract was $1 014 992.97.

Temporary Trestle on North Side of Tunnel.

During the construction of the tunnel it became necessary to support the temporary tracks on the north side of the avenue by means of a temporary trestle. The work was placed under contract with E. D. Smith and Company (Contract 42), and was completed by them

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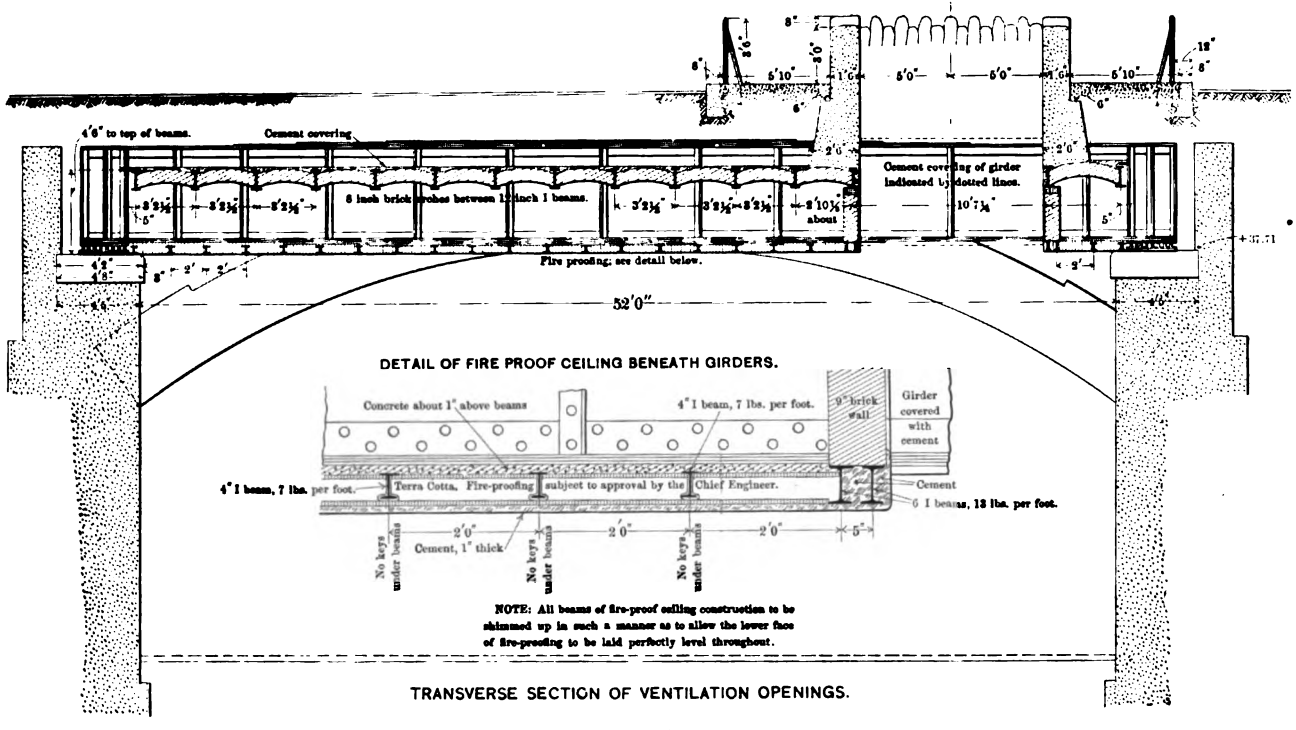

FIG. 9.

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during the tunnel construction. The work was done by unit prices, the principal ones being as follows:

Lumber in trestle, framed and erected.....	$42.00 per thousand feet, B. M.
Laying track on trestle, using old rails from the work and new sawed ties.....	1.70 per linear foot.

The total amount paid under this contract was $19 049.18.

Character and Quality of Stone.

The Contractors elected to use the following grades of stone for retaining walls and abutments, and the same were approved.

Leiperville Stone.—A light-colored gneiss from Chester, Delaware Co., Pa. It was used on the walls between Thirteenth and Fifteenth Sts., except on the north side between Broad and Fifteenth Sts.; also, on both sides between Nineteenth and Twentieth Sts., and on the abutment walls of the tunnel.

Rittenhouse Stone.—A dark-colored gneiss from Germantown, Philadelphia. Used on the north wall between Broad and Fifteenth Sts., and on the north abutment of the Fifteenth St. Bridge.

French Creek Granite.—A very dark, close-grained, heavy syenite from the Falls of French Creek, Chester Co., Pa. Used on the walls and abutments between Fifteenth and Nineteenth Sts., and on the north wall between Twenty-first St. and the tunnel portal.

Conshohocken Stone.—A medium-colored, hard, mica schist, from Conshohocken, Montgomery Co., Pa. Used on the south wall between Twenty-first St. and the tunnel; on the north retaining wall from the west end of the tunnel to Thirtieth St., and for the skewbacks in the tunnel.

Clearfield County Stone.—A medium-colored, hard, sandstone from Curwensville, Clearfield Co., Pa. Used on the abutments of the bridge at Broad St. and the south abutment at Fifteenth St., both abutments at Twentieth St., and generally for copings throughout the work.

Lumberville Stone.—A light-colored quartzite, of granitic nature, from Lumberville, Bucks Co., Pa. Used on the north and south walls, between Twentieth and Twenty-first Sts.

The physical qualities and other properties of the above-mentioned stone are given in Table No. 9.

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TABLE No. 9.—Record of Tests of Stone Used in the Construction of the Pennsylvania Avenue Subway.*

Tests Made on Eight-Inch Cubes.

Mark.	Name of stone and location of quarry.	Dimensions of specimen.	Weight per cubic foot.	Absorption, per-centage of.	Specific gravity.	Compressive Strength, in Pounds.		How Tested.		Average Compressive Strength, in Pounds per Square Inch.		Remarks.
						Ultimate.	Per square inch.	Bed or edge.	Grain.	On bed.	On edge.	
A1..	French Creek.	8 x 8 x 7 1/4	168.52	0.20	3.08	1 177 855	18 404	on bed.	none.	17 274	7 910	†Plate XXVI, No. 14.
A2..		8 x 8 x 7 1/8	168.20	0.17	3.07	1 082 640	16 144	"	"			" " " 15.
A3..		7 3/16 x 8 x 7 1/8	168.95	0.14	3.05	484 050	7 628	on edge.	"			" " " 16.
A4..		7 1/16 x 7 1/16 x 7 1/16	169.69	0.12	3.05	516 890	8 196	"	"			" " " 17.
B1..	Lumberville.	8 x 8 x 7 1/8	158.38	0.99	2.62	571 179	8 925	"	against.	14 841	8 687	Socket of compression tool broke; 629 265 lbs. replaced.
B2..		8 x 8 x 8	158.51	0.97	2.64	1 032 640	16 135	on bed.	with.			
B3..		8 x 8 x 8	158.14	1.02	2.63	935 880	14 622	"	"			
B4..		7 1/8 x 8 x 8	158.53	0.86	2.63	526 001	8 349	on edge.	against.			
B5..		8 x 8 x 8	157.40	1.08	2.63	880 971	13 765	on bed.	with.			
C1..	Rittenhouse.	8 x 8 1/16 x 7 1/8	176.05	0.15	2.84	1 174 698	18 211	on edge.	against.	11 636	13 984	Plate XXVI, No. 10.
C2..		8 x 8 x 7 1/8	174.98	0.15	2.83	871 290	13 614	on bed.	with.			" " " 11.
C3..		8 1/16 x 8 x 7 1/16	176.16	0.12	2.84	629 265	9 756	on edge.	against.			" " " 12.
C4..		7 1/8 x 8 1/16 x 7 1/8	176.18	0.13	2.84	613 180	9 657	on bed.	with.			" " " 13.
D1..	Curwensville.	8 x 8 x 8	143.11	2.61	2.44	542 136	8 471	"	"	7 513	4 468	
D2..		8 x 8 x 8	142.28	2.54	2.44	248 479	3 888	on edge.	against.			
D3..		8 x 8 x 8	146.31	1.75	2.37	322 700	5 042	"	"			
D4..		8 x 8 x 8	145.33	2.04	2.39	325 927	5 093	on bed.	with.			
D5.		8 x 8 x 8	145.18	2.58	2.43	574 406	8 975	"	"			
E1..	Leiperville.	8 1/16 x 8 x 7 1/16	165.84	0.19	2.69	564 725	8 755	"	against.	9 505	6 426	Plate XXVI, No. 6.
E2..		7 1/16 x 7 7/8 x 7 3/8	164.58	0.21	2.68	408 375	6 447	on edge.	with.			" " " 7.
E3..		7 1/16 x 7 1/16 x 7 1/16	165.00	0.21	2.69	635 719	10 254	on bed.	against.			" " " 8.
E4..		8 1/16 x 8 x 7 3/8	164.90	0.20	2.68	413 056	6 404	on edge.	with.			" " " 9.
F1..	Conshohocken.	8 1/16 x 8 x 8	176.10	0.17	2.90	522 774	8 105	"	against.	10 417	7 582	Plate XXVI, No. 1.
F2..		7 1/16 x 8 1/16 x 8	177.05	0.16	2.91	445 326	6 958	"	"			" " " 2.
F3..		8 x 8 x 7 1/16	178.90	0.15	2.92	580 860	9 076	on bed.	with.			" " " 3.
F4..		8 1/16 x 8 1/16 x 8	176.88	0.17	2.93	509 866	7 844	"	"			" " " 4.
F5..		7 1/16 x 8 x 8 1/16	177.44	0.16	2.92	910 014	14 331	"	"			" " " 5.

* Crushing Tests made at Phœnixville, Pa.

† Crushed on one side.

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Excavation of the Core. The Bridges.

Upon the completion of the retaining walls and bridge abutments upon the north and south sides of the subway, all railroad travel was diverted to Hamilton St. (except the connection with the grain elevator and Bement, Miles and Company, on the south side at Twentieth St., which was maintained over the construction tracks used by the contractors), and the avenue was given over in sections to the work of removing the core, building the bridge piers between the main walls, and constructing the bridges. This work was embraced in Contract 14, which extended from the west side of Thirteenth St. to the east portal of the tunnel. The work covered the completion of the bridge abutments, the low ends of the inside walls of the several inclines, the bridge piers, the construction of the bridges (except railings), the grading and paving of their approaches, the railings on the retaining walls not completed on other contracts, paving all adjacent driveways and sidewalks, the construction of the drainage ducts in the subway, and the preparation of the subgrade for the permanent tracks.

All the work was done at unit prices, except that a lump sum was bid for all the bridges from Broad to Twenty-first Sts., inclusive, and including, also, the paving of the decks and the approaches.

The bridges at Sixteenth, Seventeenth, Eighteenth, Nineteenth and Twentieth Sts. were erected upon the core as false-work, the intermediate piers usually being constructed in pits. Travel was maintained in all cases. At Twenty-first St., travel was cut off, and the piers were built and the bridge erected after the excavation was made. At Fifteenth St., travel was maintained by means of temporary Howe trusses; then the excavation was made, the piers were built and the bridge erected. At Broad St., half the street was closed to travel at a time; then the excavation was made, half the masonry was built and half the bridge erected and paved.

The excavated material, which was paid for by the cubic yard, without classification, including the cost of maintaining travel, was taken through the subway to the west, and, until the completion of the tunnel, was run up an incline at the east end of the tunnel and taken out over the temporary tracks. The contractor had an arrangement with the railroad company by which he loaded cars furnished by the company, and delivered them at some point agreed upon on the

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temporary tracks, and unloaded them after the railroad company had hauled them to the dump.

The greater part of the excavation was taken out by means of steam shovels, of which the contractor usually had three at work. About 500 000 cu. yds. were used by the railroad company to fill the approaches to the Delaware River Bridge at Yardley, on the New York Division. The next largest amount was used at Woodlane, on the Reading Division, a point on the Schuylkill River, below Conshohocken, Pa., where it was used to make a railroad yard. A considerable quantity of the rock was deposited on the Atlantic City Division, on the meadows between Pleasantville and Atlantic City, N. J., and the remainder was delivered at scattered points.

The highway bridges over the subway are all of the plate-girder, deck type, with solid steel floors carrying improved paving. The clearance over the main railroad tracks was 20 ft.: over the side tracks, 15 ft. Over the main tracks at the north end of the Broad St. Bridge a special clearance of 18 ft. was agreed upon, so as to prevent an undue change of the street grade. Even with these low clearances it was necessary to change the elevation of Broad St. by 4.5 ft.; of Fifteenth St., by 5.1. ft., and of Eighteenth St., by 2.2 ft.

In all cases the metal curb of the bridge is a continuation of the stone curb of the approaches, and the railings are upon the building lines, thus making the entire bridge structure below the plane of the intersecting street.

Except upon the bridge at Fifteenth St., where there is a grade of 3.96%, and where the paving is vitrified brick, the roadways are paved with sheet asphalt, and, in all cases, the sidewalks are granolithic. In all bridges, expansion is provided for by means of a specially designed joint.

The bridges at Fifteenth, Sixteenth, Seventeenth, Eighteenth, Nineteenth and Twentieth Sts. carry trolley tracks. The rail is a 6-in., 71-lb. section, laid upon longitudinal wooden stringers placed in the troughs of the Z-bar floor, for the purposes of insulation. The rail joints are "cast welded." In addition to the cast weld a $\frac{3}{4}$-in. return copper cable is placed between the rails and connected with them every 10 ft. to assist in making perfect insulation.

Upon the steel deck of the driveway was placed bituminous concrete to a depth to cover the rivet heads on the upper part of the

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Z-floor section, in the gutters, and following the crowning of the street. The asphalt wearing surface consisted of a binder coat, 1 in. in thickness, covered by the wearing surface proper, 2 ins. thick. The troughs of the sidewalks were filled with Portland cement concrete finished with a granolithic wearing surface. Inlets were placed above all expansion joints, and connected with the sewers beyond the abutments.

The loads used in the design of all bridges on the lines of intersecting streets consisted of the following:

1st. The dead load;

2d. A uniform live load of 100 lbs. per square foot;

3d. The driveways were tested with a concentrated load of 40 tons equally distributed on 4 wheels of 6 ft. gauge, spaced 20 ft. between axles;

4th. All sidewalk details were proportioned for a local loading of 110 lbs. per square foot.

All railroad bridges were designed under the standard specifications of the Philadelphia and Reading Railroad Company.

Complete detailed (not shop) drawings were prepared for all the highway bridge work, while nothing but the general lines of the railroad bridges was given for the purpose of bidding. Fig. 10 shows a typical drawing of three sections of a highway bridge.

Generally, the sidewalks were carried upon overhanging cantilever brackets, mainly on account of the additional headroom offered where crossing an inclined track leading to some industrial establishment. At Twentieth, Sixteenth and Broad Sts., a line of girders carried the outside of the sidewalk and the railing.

All steel was specified to be made by the "open hearth" process. When made in an "acid furnace," the phosphorus was required not to exceed 0.07%; if made in a "basic furnace" the limit was 0.04 per cent. The metal was required to show on a standard test piece the following results: Tensile strength, 57 000 to 65 000 lbs. per square inch; elongation in 8 ins., 25%; elastic limit, 54% of the tensile strength. For plates over 16 ins. wide, the elongation was to be 22%, and for plates over 42 ins. wide, 20 per cent. Hot and cold-bending, and quenching tests were required.

As to painting, all steel, before leaving the rolling mill, and before being exposed to the weather, was required to be thoroughly cleaned

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from all loose black scale, and given one coat of pure raw linseed oil. Before leaving the shop it was again required to be thoroughly cleaned and a heavy coat of red lead paint was applied. Two additional coats of red lead tinted with lampblack to make a dark chocolate color were applied after erection, the inaccessible parts after erection being painted previously.

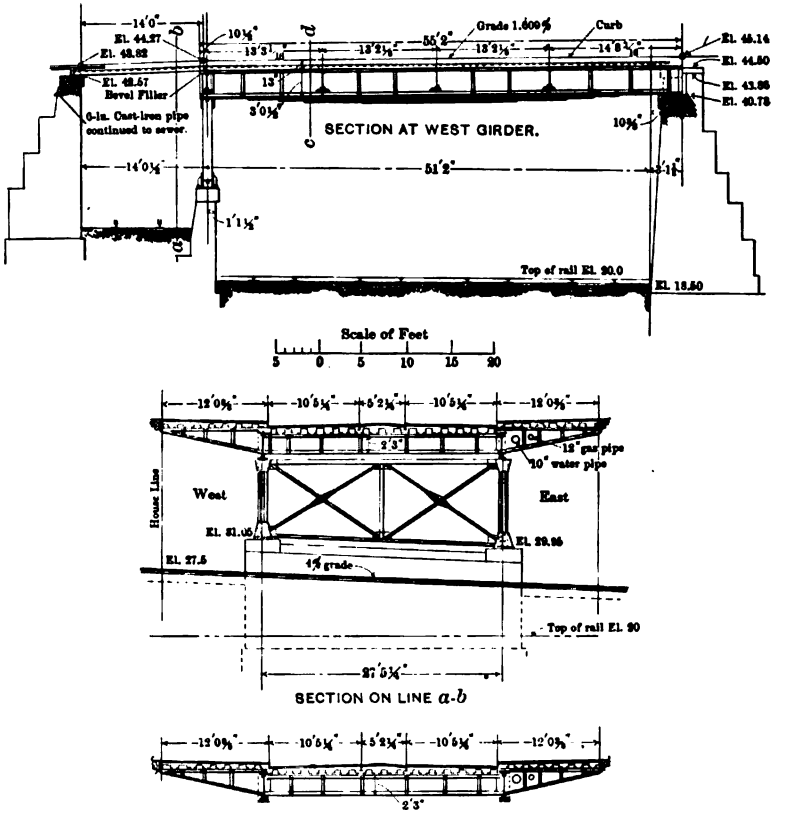

FIG. 10.

Tables Nos. 10 and 11 give the results of the tests made upon the metal in the bridges on this contract, as well as the weights of each structure and the weights per square foot of the metal-work. It will be noticed that, in the tables of the tests, averages of all the tests and analyses are given.

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TABLE No. 10.—AVERAGE PHYSICAL AND CHEMICAL TESTS OF STEEL. (BASIC OPEN-HEARTH.) CONTRACT 14.

Location of bridge.	No. of tests.	Elastic limit.	Ultimate strength.	Elongation in 8 ins.	Reduction of area.	Carbon.	Phosphorus.	Manganese.	Sulphur.
Specification requirements.	....	0.54 ult.	57 000 to 65 000	25.00	.....	.....	0.04	....	.....
Broad St.	136	36 310	60 720	29.23	56.58	0.17	0.027	0.43	0.041
Fifteenth St.	95	35 980	60 920	29.55	56.60	0.17	0.029	0.42	0.042
Driveway west of 15th St.	17	36 470	60 490	30.86	57.30	0.16	0.023	0.44	0.036
Sixteenth St.	42	37 290	60 330	29.85	56.80	0.16	0.026	0.43	0.040
Seventeenth St.	40	35 790	61 390	29.33	58.59	0.18	0.026	0.41	0.040
Eighteenth St.	37	36 440	63 550	29.54	57.90	0.17	0.032	0.43	0.044
Driveway west of 18th St.	23	37 510	60 150	29.11	58.00	0.15	0.028	0.42	0.041
Nineteenth St.	27	35 900	61 170	29.97	58.70	0.16	0.033	0.41	0.040
Twentieth St.	31	35 560	61 180	29.60	57.50	0.17	0.032	0.41	0.040
Twenty-first St.	67	34 860	60 380	31.40	55.30	0.16	0.027	0.45	0.037
Averages.	515	36 210	61 020	29.84	56.80	0.165	0.029	0.42	0.040

The following standards were required for linseed oil, red lead and lampblack:

"Raw linseed oil must be well settled and strictly pure, and free from resinous and mineral oils and gums.

"Red lead in oil must be of a good, bright color and finely ground. The pigment must contain at least 95% of red oxide of lead; no sample will be accepted that contains more than 2% of foreign matter that is vitrified, or that contains metallic lead; or that, when mixed with linseed oil and drying Japan without grinding, and applied in a good body to a vertical surface of iron, will not dry without running or separating.

"Lampblack in oil must be finely ground to a stiff paste. No sample will be accepted which is not equal in color to, or has a less staining power than, calcined lampblack, or that contains empyreumatic matter, or carbon black."

Drainage conduits, shown in Fig. 11, were placed generally in the center of the subway, the cross-section of the subgrade being arranged so as to throw all the water to them. They consist of an invert of concrete, semi-circular in cross-section, with diameters of 12 or 15 ins., as required, and placed upon a sufficient grade to carry off all the water falling upon the subway, or from adjacent buildings, to a silt basin adjacent to the deep sewers on the intersecting streets. The top of the conduit is covered with flagstones placed on top of brick

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TABLE No. 11.—Tabulated Data as to Highway Bridges (Contract 14).

Location.	Number of spans.	Length of spans.	Total length.	Width of driveway.	Width of sidewalks.	Total width.	Area of steel-work, in square feet.	Total weight of steel, in pounds.	Weight of steel, in pounds per square foot.
Broad St.....	5	33 ft. 7 1/8 ins. 61 " 2 " 61 " 2 " 61 " 2 " 38 " 3 " 40 " 0 "	255 ft. 4 1/8 ins.	69 ft. 2 1/2 ins.	2 at 22 ft. 0 3/8 ins.	113 ft. 3 3/4 ins.	29 772	2 265 062	76.4
Fifteenth St.....	6	13 " 10 1/2 " 38 " 0 " 67 " 11 1/2 " 67 " 11 1/2 " 14 " 3 "	237 " 0 "	26 " 0 3/4 "	2 " 12 " 0 3/8 "	50 " 1 1/2 "	12 125	817 825	67.4
Sixteenth St.....	2	14 " 11 " 38 " 11 "	57 " 10 "	26 " 0 3/4 "	1 " 12 " 0 3/8 "	50 " 10 1/2 "	2 942	217 097	73.8
Seventeenth St.....	3	14 " 2 " 53 " 1 11/16 " 9 " 8 "	77 " 0 "	26 " 0 3/4 "	2 " 12 " 0 3/8 "	50 " 1 1/2 "	3 743	253 410	67.3
Eighteenth St.....	3	13 " 10 1/2 " 14 " 2 " 52 " 2 "	80 " 2 1/2 "	26 " 0 3/4 "	2 " 12 " 0 3/8 "	50 " 1 1/2 "	4 084	258 875	64.1
Nineteenth St.....	2	14 " 0 1/2 " 51 " 2 "	70 " 2 "	26 " 0 3/4 "	2 " 12 " 0 3/8 "	50 " 1 1/2 "	3 516	220 987	62.8
Twentieth St.....	1	54 " 7 1/2 "	54 " 7 1/2 "	26 " 0 3/4 "	2 " 12 " 0 3/8 "	50 " 1 1/2 "	2 995	201 575	67.3
Twenty-first St.....	4	15 " 0 " 77 " 1 3/8 " 55 " 5 1/2 " 42 " 5 1/2 "	190 " 4 "	26 " 0 3/4 "	2 " 12 " 0 3/8 "	50 " 1 1/2 "	9 589	659 239	68.1

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side-walls laid with open joints, the top of the flag being designed so as to be from 12 to 24 ins. below the top of the rail. Manholes with cast-iron frames and covers are placed every 100 ft., to afford means of cleaning. The depth of the brickwork is constant, stopping at subgrade; the variable depth, required on account of the grade of the conduit, being made in the concrete work. These drains have proved to

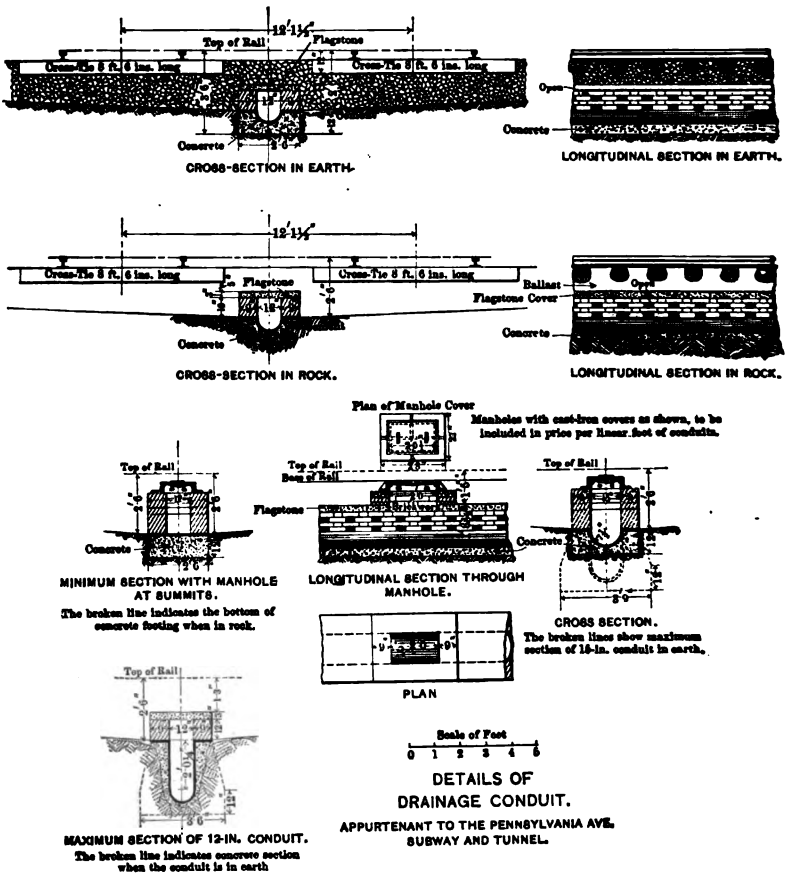

FIG. 11.

be very effective. They were paid for at a price per linear foot for each diameter, including manholes and appurtenances. As the grades were shown on the contract drawings, there was no uncertainty as to bidding.

The principal cost of the several more important items is shown in Table No. 12.

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TABLE No. 12.

Excavation	$0.59 per cubic yard.
Rubble masonry, above neat line	5.75 " " "
Ashlar masonry, in piers	15.00 " " "
Rubble masonry, below neat line	5.00 " " "
Coping, 10 x 30 ins.	1.25 per linear foot.
Bridges	$98 000.00 lump sum.
Additional steel-work	0.0325 per pound.
Railing No. 4	1.60 per linear foot.
Railing No. 6	0.95 " " "
Railing for Broad St. Bridge	23.00 " " "
Granite block paving, sand base	2.60 per square yard.
12-in. drainage conduit, in earth	1.80 per linear foot.
12-in. drainage conduit, in rock	1.90 " " "
15-in. drainage conduit, in earth	1.90 " " "
15-in. drainage conduit, in rock	2.00 " " "

The total amount paid under this contract was $736 038.92. The contractors were Messrs. E. D. Smith and Company, for whom Mr. George Rice was Engineer. Work was begun on April 15th, 1897, and was finished on August 21st, 1899. The length of time consumed does not by any means represent the time occupied in actual construction. On account of the nature of the work, it was necessary to begin on different portions of it as the other contracts were completed.

The excavation of the material between Twentieth St. and the east portal of the tunnel occupied practically 13 months. Nearly all this material was rock which was rather difficult to remove.

Bridge Railings.

The ornamental railing for the bridge at Broad St. was included in the contract for the bridges. A contract was entered into with the North Penn Iron Company (Contract 46) to manufacture and place upon all the other bridges cast-iron railings and newels.

The work was done at unit prices, which were as follows:

Cast-iron railing, in place	$2.73 per linear foot.
Cast-iron newels	7.80 each.
Bronze name-plates	19.50 "

The total amount paid on this contract was $4 941.51.

Railings.

On the contracts for the construction of retaining walls, prices were generally asked for railings Nos. 4 and 6, as shown on Fig. 12. Where the face of the wall was on the building line the gas-pipe railing, No. 6, was used throughout. Where a driveway or footwalk adjoined the wall, No. 4 was used.

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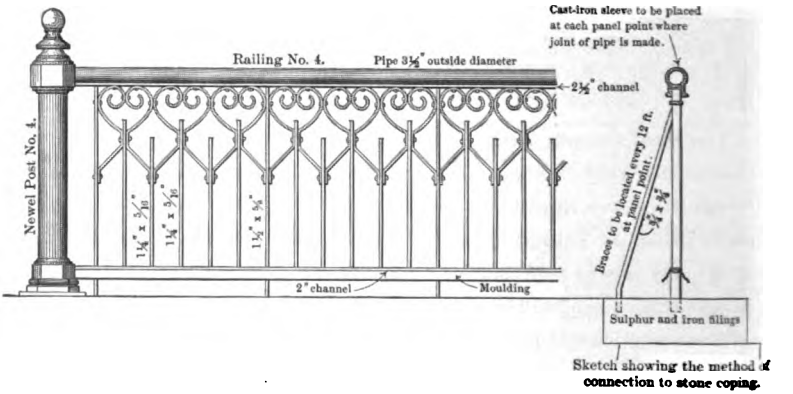
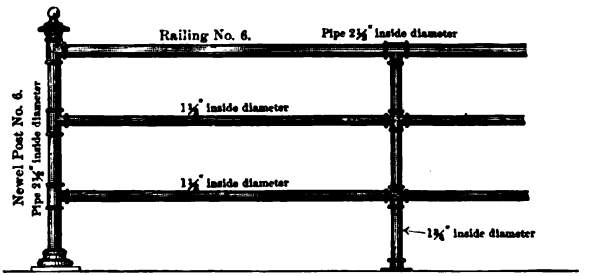
Protecting Bridges.

In 1894, when the abolishment of grade crossings on Pennsylvania Ave. was first seriously considered, the City and the railway company jointly rebuilt a bridge on the line of Girard Ave. over Pennsylvania Ave., to the west of the subway work, with a clearance of only 17.5 ft.



Sketch showing the method of connection to stone coping.



DESIGNS OF RAILINGS FOR PENNSYLVANIA AVE. SUBWAY AND TUNNEL.

Fig. 12.

over the tracks. It was decided to encase the entire metal-work on the underside of the bridge with tongued and grooved white pine boards, and to paint them with asbestos paint.

Five years after the bridge had been erected, an examination of the metal-work was made, which showed that the original paint was prac-

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tically intact, and that the paint on the outside of the wood sheathing immediately over the main tracks was still in place and the wood in good condition.

In view of this experience, and upon the completion of the bridge-work in the subway, it was decided to thoroughly clean all the metal-work, give all the bridges that crossed streets two coats of white lead body paint and to give all bridges over the subway a coat of red oxide of iron paint, and then encase all the last-named bridges with single-faced, tongued and grooved white pine fencing, in widths not to exceed 4 ins. After the completion of the woodwork, the protective casing was painted with two coats of asbestos paint, and three months thereafter two more coats were applied.

A contract for this work was made with Ryan and Kelley (Contract 57), which also included a final coat of paint on all the railings, both on the bridges and retaining walls; the protection of the water pipes which were suspended from the bridges, and a number of items of general cleaning up. (See Figs. 13 and 14.)

The water pipes on straight sections were cased as follows:

1st. One layer of No. 2 asbestos roofing and insulating felt;
2d. One layer of 1-in. hair felt;
3d. One layer of No. 2 asbestos roofing and insulating felt;
4th. One layer of 1-in. hair felt; each layer to be wrapped with tenfold sail twine;
5th. One layer of resin-sized sheathing;
6th. One layer of three-ply "A" asbestos roofing, secured with No. 17 galvanized-iron wire.

To cover the bells and lapping over the straight sections, the following was done:

7th. One layer of No. 2 asbestos roofing and insulating felt;
8th. One layer of 1-in. hair felt;
9th. One layer of resin-sized sheathing;
10th. One layer of canvas with the ends drawn to make a snug fit against the asbestos roofing on the straight sections of the pipe; outside of the canvas to be painted with waterproof composition.

Three 6-in., one 10-in., and one 20-in. pipe were thus covered; the price bid per linear foot being irrespective of the diameter. The

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work on this contract was done under unit prices, of which the following are the most interesting:

Painting of bridges	$4 100.00
Protection of bridges, except the last two coats of paint	89.00 per thousand feet, B. M.
Last two coats of paint	1 790.00
Painting bridge railings, one coat	0.12 per linear foot.
Painting No. 4 railings	0.09 " " "
Painting No. 6 railings	0.06 " " "
Covering water pipes on bridges	1.50 " " "

The total cost of the work done was $27 517.95.

Work on, and East of, Thirteenth Street.

The entire work of altering the grades of Thirteenth, Callowhill, Hamilton, Buttonwood and Twelfth Sts., together with the elevated portion of the railroad over and east of Thirteenth St. (except laying the permanent tracks), was included in Contract 12, which was awarded to Messrs. P. H. Flynn and Company on July 28th, 1896. All the work on this contract was paid for by unit prices, except the steel work of the bridges, which was bid for in a lump sum.

Before work was begun the double-track railroad approach to the Reading Terminal at Twelfth and Market Sts. was carried on a wrought-iron viaduct from Callowhill St. east of Twelfth to Thirteenth and Noble Sts., and thence by a masonry approach to the grade of the old street at Broad and Noble Sts. The alignment of this part of the structure was not changed. The wrought-iron viaduct between Callowhill and Twelfth Sts. was lowered about 2 ft. at Twelfth St., and remained unchanged at Callowhill St., the grade of Twelfth St. being lowered by the same amount. The metal-work of the viaduct was blocked up, the masonry cut down, and new bearing plates were inserted, to suit the new grade. Twelfth St. is crossed by a plate-girder, half-through span, which is skewed at an angle of 66° 17′ with the street, and consists of two girders 28.5 ft. from center to center, supporting the ends of a trough-floor composed of plates and angles, the troughs being 20 ins. deep. The ballast and ties are placed in the troughs so that the distance from the top of the rail to the clearance line of the street below is a minimum.

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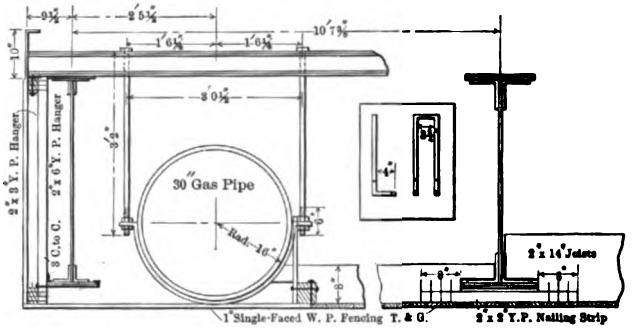
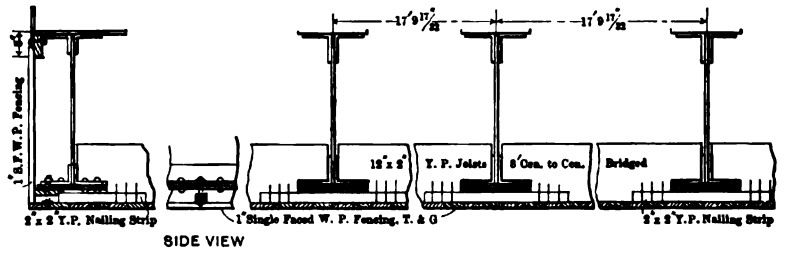
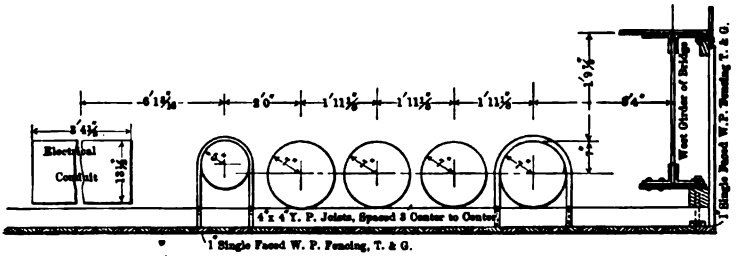


Sixteenth Street Highway and Railroad Bridge



SIDE VIEW



Sheathing: Broad Street Bridge

NOTE: At columns and newel post of railing, the 2 x 5-in. Y. P. Stringer bolted to base of railing, as shown, is to follow around the edge of base of columns and newel posts of railing. The 2" x 8-in. Y.P. nailing strips at bottom of Girder to extend far enough beyond the Y. P. stringer to allow the sheathing to be bored around column and newel posts and brought down square with base of same to catch sheathing on bottom of Bridge.

BRIDGE PROTECTION

PENNSYLVANIA AVE. SUBWAY.

Scale of Feet

0 1 2 3 4 5 6 7 8 9 10

FIG. 18.

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Between Twelfth and Thirteenth Sts. the old metal structure was entirely removed, and was replaced by retaining walls of rubble masonry with earth filling between.

Between Twelfth and Thirteenth Sts., on the south side of the tracks, is located a coal yard, the property of the Philadelphia and Reading Railway Company. This was excavated to an average depth of 10 ft. and connected with the Terminal tracks west of Thirteenth St. by means of two sidings. At Thirteenth St. the two railroad tracks which are on Noble St., and extend to the Delaware River, connect with the Terminal tracks. The bridge at Thirteenth St. has provision for a highway passageway on the north, on the line of Noble St. and six railroad tracks to the south. It is a steel, half-through, plate-girder bridge which has a solid steel floor and is very badly skewed. The steel floor rests upon the bottom chord of the plate girders. The drainage is effected with bituminous concrete, placed in the bottom of the troughs and sloped toward the center, where a hole is provided to allow the water to pass through and into a half-round gutter of skelp-iron, leading to a vertical pipe fastened to the abutment, and thence into the sewer. Ballast is placed directly over this bituminous concrete. The highway portion of the bridge is of a solid-steel trough section in the floor, covered with bituminous concrete and paved with granite blocks. The total weight of the steel in this bridge is 610 754 lbs.

The grade of Thirteenth St. at the intersection of Noble St. is lowered 13.6 ft., and this is the deepest point in the depression. From this point the new grade ascends to the north at the rate of 3.7%, reaching the grade of the old street at Nectarine St., a distance of 471 ft. On the south, the ascending grade is at the rate of 3.5%, reaching the original grade of the street at Carlton St., a distance of 388 ft. On the west side, just south of Noble St., is the depressed freight yard of the subway, the main driveway of which meets the level of Thirteenth St., and thus makes an access to the depressed yard.

To the north of Noble St. there are a number of large buildings; on the south the number is smaller. Before any steps were taken to change the grade of the street, the foundations of all structures on Thirteenth, Callowhill, and Hamilton Sts. were underpinned and carried down below the new grade of the street, generally to a depth of 3 ft. below the new pavement subgrade. On account of the impracti-

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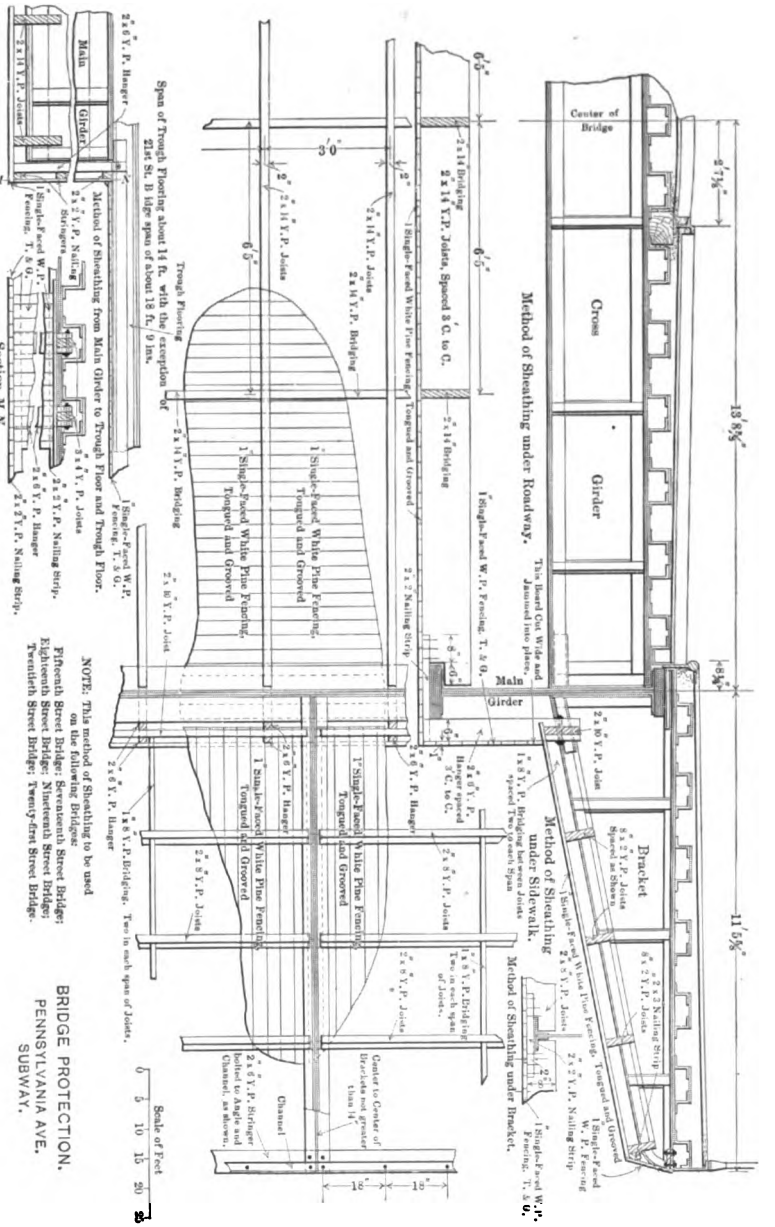

Fig. 14.

NOTE. This method of Sheathing to be used on the following Bridges:

Fifteenth Street Bridge; Seventeenth Street Bridge; Eighteenth Street Bridge; Nineteenth Street Bridge; Twentieth Street Bridge; Twenty-first Street Bridge.

BRIDGE PROTECTION. PENNSYLVANIA AVE. SUBWAY.

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cability of making any arrangements with the property owners as to the future adjustments of their buildings, the underpinning was made with rubble masonry laid in Portland cement mortar, the wall being flush with the face, and of the same thickness as the foundation, of the old building. This work was done in sections, the trench excavated in front of the buildings generally being 3 ft. wide. As fast as the underpinning was completed, a temporary sidewalk of yellow pine was placed over it, so as to interfere as little as possible with the access to the properties. This underpinning was paid for by the cubic yard, the price being $20. In this price was included all shoring, sheathing, excavation, and all appurtenances. It was very successfully carried on, without any injury whatever to the structures along the work.

Upon the completion of the underpinning, the excavation of the street was begun at a number of points. Before it was actually commenced, the street-car traffic, by agreement between the contractors and the Traction Company, was diverted to Twelfth St. by means of a loop. One of the large electric feeders of the Union Traction Company, consisting of a number of ducts surrounded by concrete, was located on the east side of the street. The Traction Company drove two rows of piles, opposite each other, on either side of the conduit, at intervals of about 12 ft., the piles being driven well below the new subgrade. Excavation was then made from pile to pile beneath the conduit, and a heavy cross-piece was placed beneath the conduit and bolted to the piles. This remained until the excavated material in the street was removed. Then a new excavation was made, a new conduit placed in it, new cables were drawn and connected up, and the old conduit was broken up and removed.

As the excavation proceeded, the new sewer was laid on the west side of the street, and all house connections were made to it. The water and gas mains were also lowered, and all connections were made to the house line. The streets were finally paved between the curb lines with granite blocks on a 6-in. concrete base, and the sidewalks generally with brick, except where some other class formerly existed.

Considerable delay was caused for some time by being unable to do away with the temporary railroad crossing on Hamilton St. When this track was finally abandoned, the travel was taken from Noble St. into the subway, and up the grade at Sixteenth St. on the temporary tracks again at Sixteenth and Hamilton Sts. The abandonment of the

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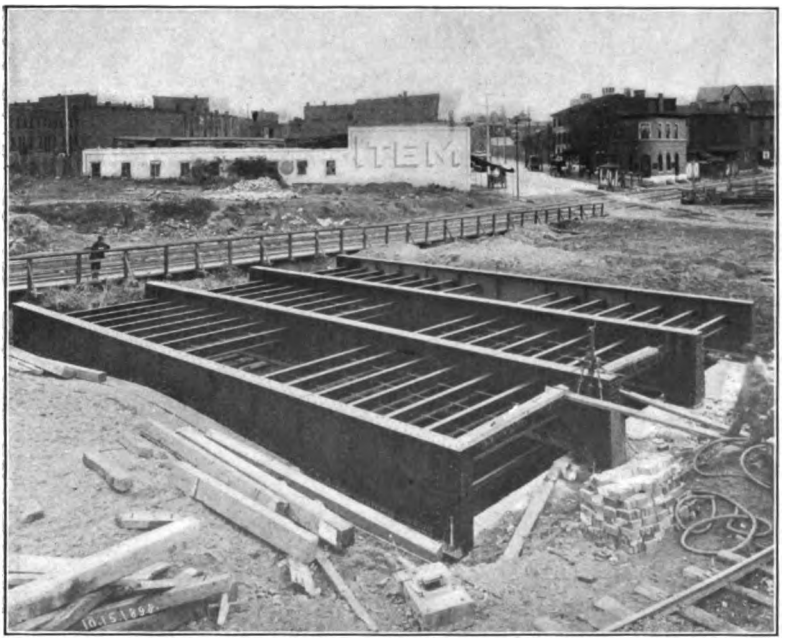

FIG. 1.—STEEL-WORK OF VENTILATION OPENINGS.

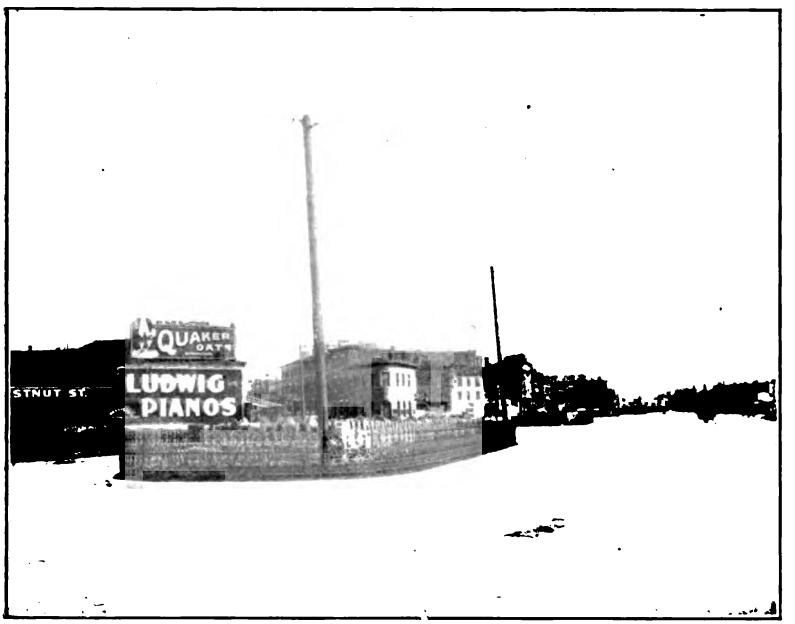

FIG. 2.—VENTILATION OPENING AT TWENTY FIFTH STREET.

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track crossing Thirteenth St. enabled the change of grade to be finished on that street and on Hamilton St., and the work completed on that contract.

The most interesting prices on this contract are shown in Table No. 13.

TABLE No. 13.

Excavation (unclassified)	$0.56 per cubic yard.
Rubble masonry, above neat line	6.00 " " "
Ashlar abutment masonry (rubble backing)	8.62 " " "
Coping (10 x 30 ins.)	2.25 per linear foot.
Rubble masonry underpinning, including excavation	20.00 per cubic yard.
Repaving with granite block on new 6-in. concrete base	1.50 per square yard.
Paving with granite block on 6-in. concrete base	2.75 " " "
Brick sidewalk paving	1.00 " " "
Sewer, 2 ft. 9 ins. diameter	2.25 per linear foot.
Sewer, 1 ft. 8 ins. diameter	2.00 " " "
Sewer, 2 ft. 6 ins. diameter	2.25 " " "
Yellow pine lumber in trestles, etc.	27.00 per M. ft. B. M.
Steel-work in bridges at Twelfth and Thirteenth Sts., and adjustment of old work	$6 695.00.

Work was started on this contract on November 12th, 1897, and final payment was made on November 28th, 1899. The total amount paid under the contract was $152 442.94.

Changes in Plan Between Sixteenth and Seventeenth Streets.

On account of the changes already referred to on Pennsylvania Ave., between Sixteenth and Seventeenth Sts., the contract with P. H. Flynn and Company was terminated on the south side of the street, and Contract 44 was executed with E. D. Smith and Company to complete the work on the revised plans. The work was paid for at unit prices, of which the following are of interest:

Tearing down old masonry	$1.00 per cubic yard.
Excavation, unclassified	1.00 " " "
Rubble masonry, below neat line	5.50 " " "
Rubble masonry, above neat line	6.00 " " "
Rock-faced ashlar, rubble backing	8.50 " " "

The total amount paid on this contract was $13 444.48.

Special Connection North of Nineteenth Street.

On the north side of the subway, between Nineteenth and Twentieth Sts., is located the works of a manufacturer of malleable and steel castings. After the plans were completed it was desired to con-

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nect these works with the subway. Plans were prepared to connect with the tracks at the subway level by means of a hydraulic lift placed upon the property of the manufacturer. This lift was provided by the owner of the property, and its cost was included in his claim for damages. It is designed to lift a single car from the new to the old level, and is operated by a single vertical hydraulic cylinder. The City made all the excavations for the lift, dug the hole for the hydraulic cylinder, walled in the whole excavation, built a bridge on the line of the sidewalk on the high level, and reconstructed the building over the lift. The work was done under contract with E. D. Smith and Company (Contract 41), the total amount paid being $19 904.20.

Changes in Water and Gas Mains.

One of the most troublesome, and, in some cases, difficult parts of the work was the readjustment of the water and gas mains which were interfered with.

Water Mains.—On account of the necessity of having the changes at all times under the control of the proper municipal authorities, arrangements were made to advertise for the pipe, specials, valves, lead, gasket, etc., and have the actual work of making the changes performed by the employees of the Bureau of Water, the several contractors, however, making the necessary excavations and doing all the back-filling. The cost of the labor performed by the Bureau of Water was kept separate from their other work. The cost of the labor of making the changes hereafter described amounted to $34 579.35. The pipe and specials furnished under Contract 35 with the McNeal Pipe and Foundry Company amounted to $14 968.70, and the lead and gasket furnished under Contracts 36 and 39 amounted to $1 428.14, making the total cost of the water-pipe changes $50 976.19.

Outside of the relaying of several small mains caused by the change of grade of Thirteenth St., and the carrying of the 6, 8 and 10-in. mains over the bridges at Sixteenth, Eighteenth and Twentieth Sts., as well as other small mains west of Twenty-second St., the principal work was as follows:

Lowering the 30-in. mains on Broad St. to pass through the piers and abutments, and beneath the tracks; lowering a 6-in. main on Seventeenth St. in the same manner; excavating two tunnels through the rock on the east and west sides of Twenty-first St., before the core of

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the subway was removed; placing a 48-in. and a 6-in. main in one, and a 20-in. and a 24-in. main in the other. These tunnels were driven at such a depth that when the pipes were laid and covered with 3 ft. of earth the top would be removed in excavating the core of the subway to subgrade, thus removing the rock from between the mains and the subgrade of the tracks.

An old line of 22-in. main on Pennsylvania Ave., laid in 1819, was removed, and a new line of 24-in. pipe was laid on the north side of the tunnel to replace it. The old pipe was found to be in remarkably good condition after 78 years of continuous service. The corrosion on the interior surface was found to displace about 4.2% of the original area.

From Twenty-second St. to the west the pipe lines cross the work over the top of the tunnel. Difficulty, however, was encountered on the line of Twenty-fourth St., Green St. and Fairmount Ave., where the mains are 48 ins. in diameter, in providing the necessary room between the top of the tunnel and the street above. The method of overcoming this difficulty has already been described.

Gas Mains.—The same general method of procedure was agreed upon with regard to the gas mains, but, before the work had advanced very far, the Gas-Works of the City were acquired under a lease with The United Gas Improvement Company, who were obligated, under their agreement, to make all the necessary changes. Under Contract 37, material to the value of $1 437.33 was furnished, but no account of the labor for making the necessary changes is available. On the line of Broad St. a 30-in. main was divided up into four 12-in. mains, and carried across the yard on the under side of the floor. The Gas Company also laid an additional 24-in. main beneath the tracks.

Permanent Tracks.

As fast as the excavation of the subway and tunnel was completed, and the subgrade prepared on the elevated portion of the work, the construction of the permanent tracks was begun by Contractors Ryan and Kelley, under Contract 30. The contract, besides covering all the trackwork of a permanent nature, included the removal of the temporary tracks and the restoration and repaving of such portions of the work as should be directed by the Chief Engineer. The trackwork proper, including the removal of the temporary tracks, was bid for in a lump sum, there being unit prices for all the work of street

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restoration. All the material in the temporary tracks became the property of the contractor for the permanent tracks, and he was required to make allowance therefor in his bid on the work.

The four main tracks were required to be ballasted with good, hard, durable, broken stone, crushed or broken into fragments which would pass a 2½-in. ring in any direction, and be perfectly free from dirt or any other admixture before being spread. It was laid to the lines shown on the drawings, with a minimum depth of 8 ins. beneath the ties at any point. The specifications allowed the use of clean, hard, vitrified, blast-furnace slag, in place of the stone, in the yards and on sidings, but very little of it was used on the work, on account of the difficulty of procuring a satisfactory quality.

The four main tracks were laid with first-class, white oak ties, laid sixteen to each 30-ft. rail. For the sidings and yard tracks, the specifications allowed the use of 20% of second-class ties. First-class ties were 8½ ft. long, 7 ins. thick, and not more than 14 nor less than 7 ins. wide. Second-class ties were to be the same as first class, but not less than 6 ins. face.

The rails of the four main tracks weigh 90 lbs. per yard, and the sidings and yard tracks, 80 lbs.; the sections for these weights being the standards of the Philadelphia and Reading Railway Company. The chemical composition of the rails is shown in Table No. 14.

TABLE No. 14.

Elements.	80-lb. rails.	90-lb. rails.
Carbon	0.55 to 0.60	0.60 to 0.65
Silicon	0.15 to 0.20	0.15 to 0.20
Manganese	1.10 to 1.30	1.15 to 1.35
Phosphorus	0.06	0.06
Sulphur	0.069	0.069
Rails having carbon below, will be absolutely rejected	0.53	0.58
Rails having carbon above, will be absolutely rejected	0.65	0.67

The drop-test required that one rail butt be taken from each heat, and that 90% of the specimens must stand without breaking, and that in cases where the broken butts do not show an elongation of 4% per inch under greatest tension, the rails of the heat are to be held until a piece can be cut from one and tested under the drop. Then, if it fails to show the 4% elongation before breaking, the entire lot of

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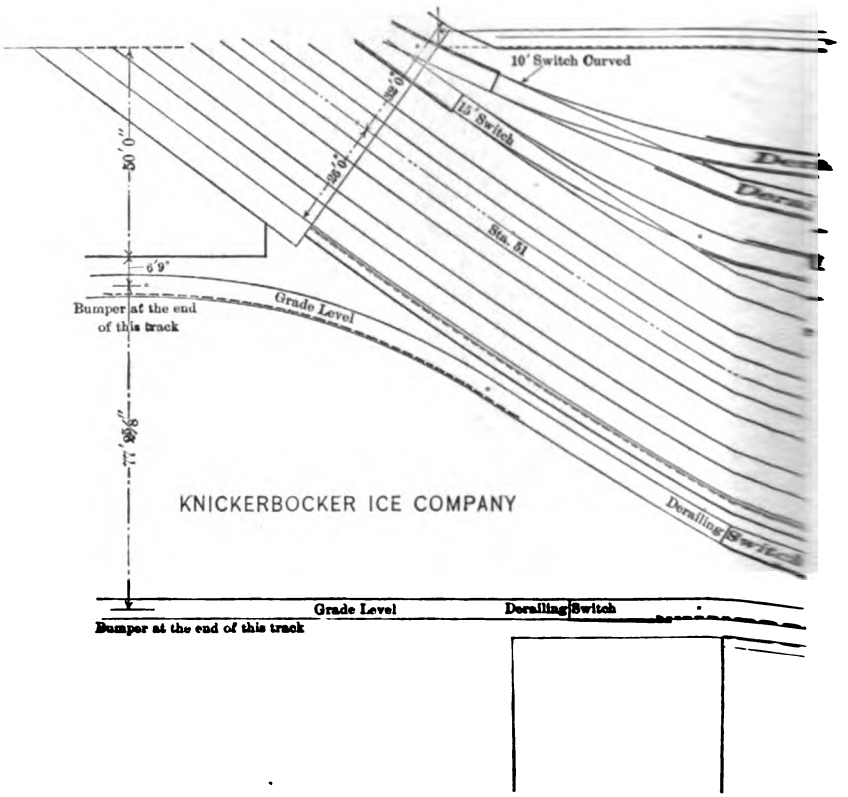

PLAN OF PERMANE
IN THE TWENTIETH S
SHOWING LOCATION
SWITCHES, ET

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rails made from that heat shall be subject to rejection. The loads, height of drop, etc., were as follows:

Weight per yard.	Height of drop.	Distance of supports.
80 and 90 lbs.	20 ft.	3 ft.

All frogs, switches, spikes and other track details were made from the standard patterns and drawings of the railway company.

Before any material was ordered by the contractors, the City prepared working plans of the tracks, showing the definite location of all frogs, switches, special ties, etc. These plans were used by the contractor in ordering, by the engineers in the field in laying out the work, and by the contractors for the interlocking switches and signals. They proved to be of great value in expediting the work and preventing errors. Plate XVIII shows this work in the Twentieth St. yard.

The tracks on the north side of the yard, between Thirteenth and Sixteenth Sts., as well as the two main tracks between these streets, were first completed, and, as fast as the subgrade was finished, two tracks were extended to the west through the subway and tunnel to the end of the work at Thirtieth St. The railway company then began carrying some of its regular freight business on these lines, and the work of removing a portion of the temporary tracks was begun. An effort was made to connect all the industrial establishments on the south side of the line with these tracks, so as to facilitate business. The last of the track laying was done in the freight yard at Twentieth St., and on the upper level of Sixteenth and Hamilton Sts. The last of the temporary tracks removed was on the high level between Twenty-sixth and Thirtieth Sts.

Hamilton St., for most of the distance between Broad St. and Pennsylvania Ave., was originally paved with cobble, and as it was decided during the construction of the work to repave it with new granite blocks, for which there was no price in this contract, arrangements were made to advertise for it under Contract 57.

Work was begun on this contract on August 1st, 1898, but, on account of the conditions existing on the other contracts, which prevented any large or continuous force being employed, it was not completed until September 9th, 1900.

The most interesting prices on the contract are as follows:

Track laying, with all appurtenances.....$209 453.00
Asphalt block repaving.....$2.50 per square yard.
Sheet asphalt paving, 6-in. concrete base.. 2.60 " " "

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The total amount expended under the contract was $238 036.47.

The more important quantities embraced under the lump-sum bid are as follows:

27 584.21 ft. of single track with 80-lb. rail, ballasted.
41 244.26 " " " " 90 " " "
1 732.22 " " " " 80 " " on trestle.
1 560.00 " " " " 80 " " on timbers in engine-house.
2 226.01 " " " " 88 " tram rail laid in paved streets.
54 448 ft. of temporary track were removed.

Restoration of Hamilton Street.

The contractor for the permanent tracks, as part of his contract, was required to remove the temporary tracks from Hamilton St., and a contract was made with Michael O'Rourke (Contract 56) for restoring the paving, resetting old and furnishing new curb, reconstructing inlets and manholes, and replacing the industrial track connections crossing the street. The work was done at the following unit prices:

New granite block paving, 6-in. concrete base..$2.40 per square yard.
Resetting old curb..... 0.25 " linear foot.
New crossing stones, 20 ins. wide..... 1.00 " " "

There were also a large number of other items covering all classes of reconstruction, none of which is of special interest.

The total cost of the work was $36 541.70.

Guard Rails on 5% Inclines.

A contract, known as Contract 51, was entered into with Wilkins and Kuemmerle, which consisted in furnishing and laying additional guard rails on the metal approach from Twelfth to Callowhill Sts.; on the 5% grades beneath the bridges, and in some special cases in the yards. These guard rails, when planed to give 2½ ins. between the heads, weigh 80 lbs. per yard. They were furnished at a cost of 40 cents per linear foot, while the same, unplaned, were furnished at 50 cents per foot. The total cost of this work amounted to $5 220.10.

Freight Bumpers.

The freight bumpers were of the Philadelphia and Reading standard, and were furnished under Contract 53 with Ryan and Kelley. They were of three kinds, and the prices for the bumpers, delivered and set in place, were as follows:

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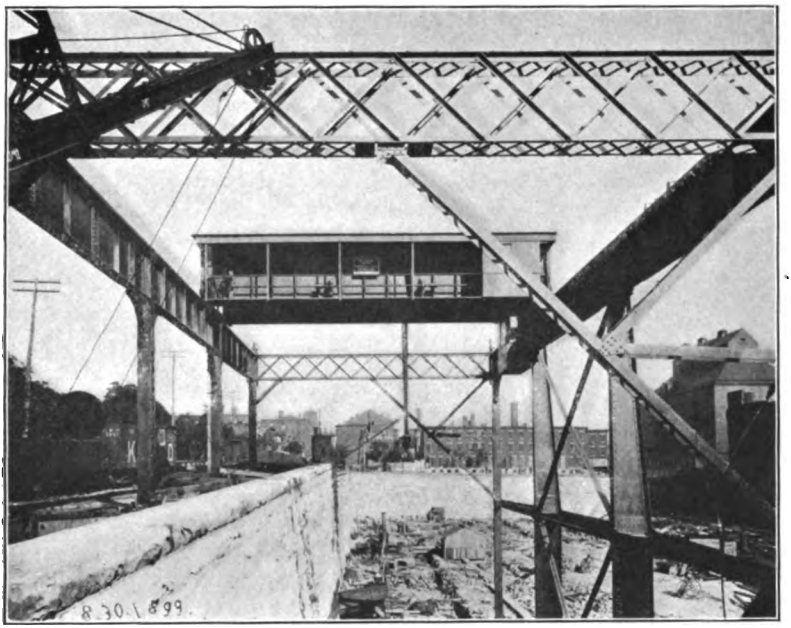

FIG. 1.—ELECTRIC CRANE, TWENTIETH STREET YARD.

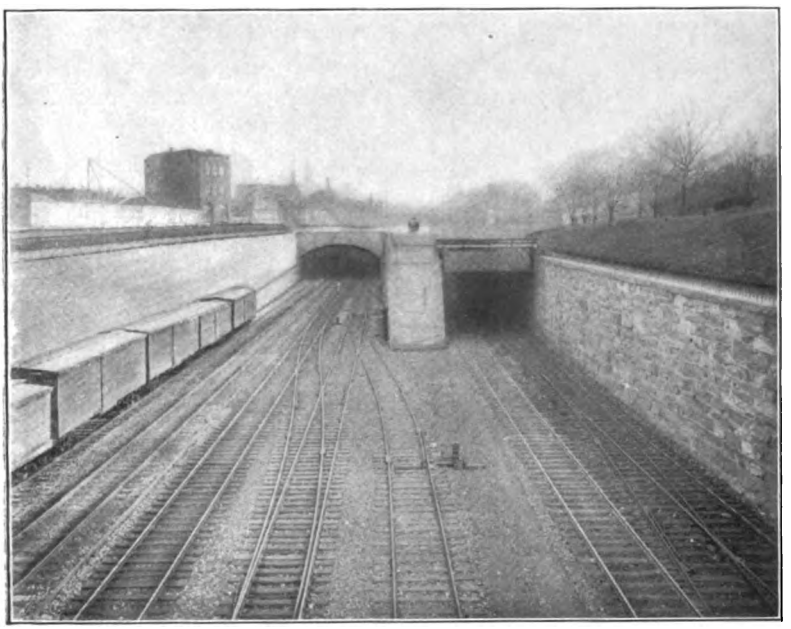

FIG. 2.—WEST PORTALS OF SUBWAY AND BALTIMORE AND OHIO TUNNELS.

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Standard high bumpers, 28 pieces, each.....				$200.00
"	"	"	on trestle, 4 pieces, each...	156.60
"	"	"	against walls, 6 pieces, each.	80.00

The same contract covered three special steel bumpers on the bridge at Sixteenth St., at a total cost of $1 525, and some additional items in connection with finishing the work at Sixteenth and Twenty-first Sts. The total payments for all the work on this contract amounted to $11 551.

Interlocking Switches and Signals.

The main tracks, from the connection with the tracks leading to the Reading Terminal to the east portal of the tunnel, are controlled by means of a complete system of interlocking switches and signals. This work was bid for in a lump sum, bids being asked upon a general specification requiring the submission of a general plan and detailed specifications with the bid. The work was awarded to Messrs. E. D. Smith and Company, who presented a plan and specifications prepared by the National Switch and Signal Company, of Easton, Pa. In the original specifications, either a manual or an electro-pneumatic system was stated to be satisfactory.

The scope of the work is best indicated by the description of the signal towers, which is as follows:

"Tower No. 1 on the west side of Broad St. will operate all switches and signals from the east of Thirteenth St. to a point west of Fifteenth St. This tower contains 51 active levers and 5 blanks (for future extensions) for the operation of—

35 switches,
5 movable-point frogs,
1 Scotch block,
34 facing-point locks,
32 signals.

"From the point west of Fifteenth St. to a point east of Twentieth St., switches will be operated from Tower No. 2, which is on the line of Seventeenth St. in a recess in the north abutment of the bridge. This tower contains a machine with 51 active levers and 5 blanks for the operation of—

35 switches,
5 movable-point frogs,
1 Scotch block,
11 locks,
34 facing-point locks,
32 signals.

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"From the point east of Twentieth St. to the east portal of the tunnel, all the switches and signals are operated from Tower No. 3, which is located immediately east of Twenty-first St., which contains a machine having 75 active levers and 5 blanks for the operation of—

46 switches,
7 movable-point frogs,
10 locks,
50 facing-point locks,
44 signals.

The system used is a manual one, and the following are some of the principal features. The pipe line carriers were spaced 8 ft. apart, the wire pulleys 24 ft. apart.

Machines.—The levers are pivoted on a common center, spaced 5 ins. from center to center. The top plates and girders are made in sections for four and eight levers. The switch and lock levers are in the center. The locking is preliminary latch locking; the first act in the operation of the lever is to lock all conflicting levers.

The main pipe lines are 2¾ ins. between centers. The pipe is of iron or steel, 1 in. in diameter. All lengths of pipe of more than 100 ft. are compensated, and additional compensation is provided for all lengths of more than 700 ft. The compensator foundations are of oak, with concrete.

Detector bars are provided, as shown on the detailed plans, 45 ft. long, with 12 clips to each bar which rises ¾ in. above the top of the rail.

The wire is single-strand, No. 9, galvanized steel, the tensile strength being not less than 1 300 lbs. and the elongation from 4 to 8 per cent.

The signals are of standard semaphore type.

The contract price for all the foregoing work, completed and in place, was $85 000. The work was begun at the tower at Broad St., and was carried on as fast as the construction of the permanent track allowed. Work was begun on August 25th, 1898, and the final payment was made on July 5th, 1900. The total amount paid was $86 376.

In addition to the work of the interlocking, under this contract, the entire length of the subway is equipped with Hall automatic block signals, which were separately contracted for by the Philadelphia and Reading Railway Company.

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Telegraph Service.

During the construction of the work, the overhead telegraph wires were removed, a temporary cable was purchased, under Contract 38 with the Western Electric Company, and was laid by the Electrical Bureau in a City duct from Twelfth and Callowhill Sts. to Thirty-first St. and Girard Ave., entirely outside of the line of the work. The cable consisted of 40 wires of No. 16, B. & S. gauge, and 18 wires of the same gauge, twisted in pairs, all properly insulated and lead covered. Connections were made at the railroad offices at Sixteenth and Hamilton and at Twentieth and Hamilton Sts. There were 15 958 ft. of temporary cable, costing $4 468.24.

The permanent telegraph and telephone line was laid below ground in the subway in a three-duct creosoted conduit. As the capacity asked for by the railroad company was in excess of that which they originally had, a special agreement was made whereby the railroad company furnished the ducts, and the City, under the subway account, furnished the cable, the junction boxes, the manhole frames and the covers, and laid the cable by day's labor on the pay rolls of the Electrical Bureau.

The permanent cable consists of 10 wires of No. 14, B. & S. gauge, twisted in pairs, and 40 wires of No. 14, B. & S. gauge, insulated with manila rope paper, and lead covered. There were 11 272 ft. of cable, costing $5 083.68, which was furnished under Contract 55 with the Standard Underground Cable Company.

Coal Yard, Fifteenth and Callowhill Streets.

It was originally intended to construct a large coal-handling plant at Fifteenth and Callowhill Sts. The plans were modified at the suggestion of the railway company, and a coal yard was constructed under contract with Ryan and Kelley (Contract 49). The yard consists of a wooden coal trestle connecting with the high-level tracks at Sixteenth St. and Pennsylvania Ave., and provided with the necessary bins. An office and a two-story stable were included, and the yard was paved with granite blocks. The work was done under the following unit prices:

Stable	$2 700.00
Office building and scale	1 300.00
Track and trestle	2 950.00

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Excavation, unclassified	0.75 per cubic yard.
Rubble masonry, above neat line.....	5.50 " " "
Rubble masonry, below neat line	5.00 " " "
Paving with granite block, sand base.....	1.75 " square yard.
Derailing switches in subway.....	50.00 each.

The total amount paid was $19 401.61.

Locomotive Coaling Station, Twentieth Street Yard.

Immediately to the west of the engine-house in the Twentieth St. yard, and located so as to span all the tracks leading therefrom, as well as the four main tracks in the subway, there is a locomotive coaling station, constructed under Contract 27 with the Link Belt Engineering Company.

It consists essentially of a storage pocket for coal and ashes, and a coaling bridge.

The storage pocket, 22 ft. wide, 52 ft. high and about 136 ft. long, is divided into two compartments. One compartment is for coal and the other is a pocket, 9 ft. 4 ins. wide and 20 ft. long, for ashes. The pockets are carried on steel posts supported on masonry piers. The framework is of steel sheathed on the inside with 3-in. closely matched, yellow pine, and on the outside with No. 22, galvanized, corrugated iron. The ash pocket is lined inside with concrete.

The coaling bridge consists of steel trusses, closed in and roofed over. It is provided with two lines of coaling tracks with turn-tables. There are six openings for discharging directly into locomotives and into two loading hoppers, each of 4 tons capacity. The coal elevator is of the gravity discharge type, with a capacity of 120 tons of coal per hour. The necessary engine power is included under the contract. The cost of the plant was $59 458.13.

Locomotive Water Supply.

The arrangements for supplying engines with water were included under Contract 50 with McParland and Butler, and consisted generally in furnishing four water columns, situated in the railroad yards, and making water and drainage connections to two others which were furnished by the railway company, which also supplied the tanks. The total cost of the work was $4 766.62. The principal items for payment on this contract were as follows:

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PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLVIII, No. 930
WEBSTER AND WAGNER
ON PENNSYLVANIA AVENUE SUBWAY.

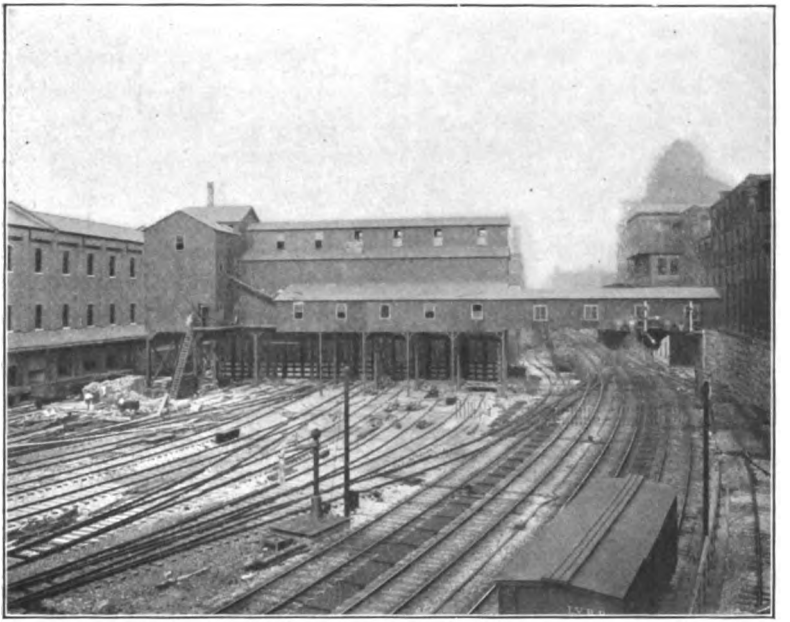

FIG. 1.—COALING STATION, TWENTIETH STREET YARD.

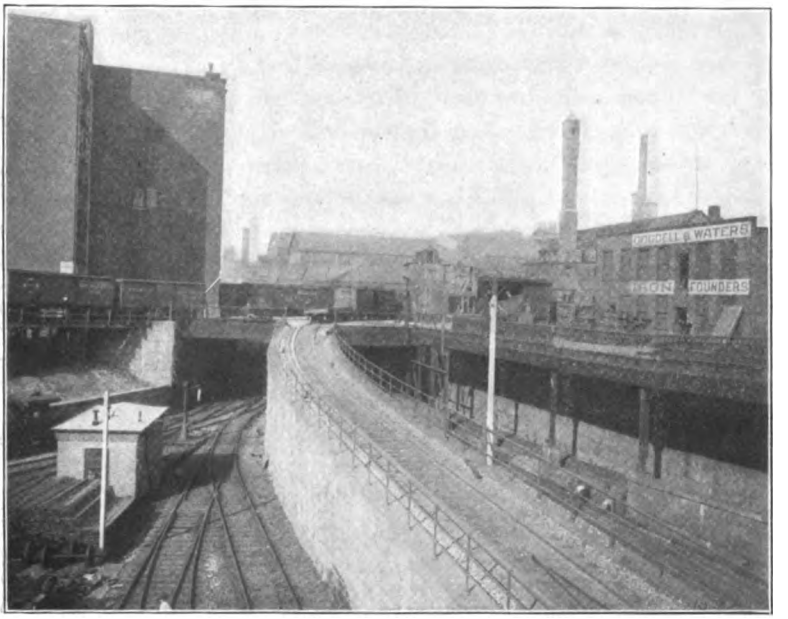

FIG. 2.—FIVE PER CENT. GRADE TO SIXTEENTH STREET.

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Excavation (not classified).....	$0.80 per cubic yard.
8-in. cast-iron pipe.....	0.47 " linear foot.
4 " " "	0.28 " " "
Water-columns, complete, each.....	$590.00

Freight-Houses.

Two terminal freight stations are included within the limits of the work: First, Broad St.; and second, Twentieth and Hamilton Sts.

Broad Street Station.—The station at Broad St. is a very important one. Before the construction of the Reading Terminal, in 1892, the main-line passenger station was located on Callowhill St., extending from Broad to Thirteenth Sts. To the north of it the freight station was located. After the abandonment of the passenger station, in 1893, the passenger building was used for freight purposes.

Upon the construction of the temporary tracks of the subway, the eastern half of these buildings was removed, and a temporary freight station at the north-west corner of Thirteenth and Callowhill Sts. was constructed, as before described. Upon its being occupied, the remaining old buildings were removed, and the excavation was made to the subway level for the western half of the block. A contract which had been entered into with Ryan and Kelley (Contract 15), was then begun, and the western halves of a double freight station were started and temporarily finished at the eastern end. As soon as it was completed and the tracks in the depressed yard were ready for operation, the remaining buildings, out to Thirteenth St., were finished.

The freight-houses, one for inbound and the other for outbound business, are of the following dimensions: One, 37 ft. wide, and 524 ft. long, and the other, 37 ft. wide and 465 ft. long. The buildings are of brick, with steel trusses and slate roofs, and are one story in height. At the Thirteenth St. end of the shorter building there is a three-story office building 43 ft. wide and 56 ft. 8 ins. long, with a cellar. Between the buildings there is a driveway 60 ft. wide, paved with granite blocks. At the east end the entrance is on a level with Thirteenth St. At the west end access to the buildings is obtained beneath the Broad St. Bridge and by means of the inclined approach, built on a 5% grade parallel to Callowhill St., reaching the street level at Fifteenth St. On the track side of each freight house there is a platform for loading and unloading freight.

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Fire-walls are built in the houses at intervals, and are provided with rolling steel shutters. Shutters of the same type are also provided on the track sides of the buildings.

The work on this contract was bid for in a lump sum for the buildings. The paving of the driveway was bid for by the square yard. The total cost of the buildings was $98 251.20; the price for the paving was $3 per square yard. The total amount paid was $108 763.23.

Twentieth Street.—There are three buildings in the depressed yard at Twentieth St., as follows: A freight-house, an engine-house, and a repair shop. These were constructed by P. McManus, under Contract 25.

The freight-house consists of a two-story brick building 33 ft. wide and 308 ft. long, with a platform on the track side in the depressed yard and wagon access on the level of Hamilton St. Fire-walls between the elevators are provided with rolling steel shutters. Shutters of the same type are provided for the doors on the track side. Steel trusses, with a slate roof, cover the building. Five hydraulic elevators are provided for raising or lowering the freight from the street level to the subway. They have platforms, 10 x 14 ft. in the clear, and each is calculated to carry a load of 10 tons. They are designed to operate at a speed of 50 ft. per minute with a moderate load, and three of them, when fully loaded, are capable of rising 33 ft., the entire distance, at the same time. They are provided with automatic stops at the top and bottom, and with safety attachments and gates. All piping, pressure tanks and pumps are provided. The latter are located in the repair shop, and are in duplicate. The price bid for this building, complete, was $74 105.

Engine-House and Repair Shop.—The engine-house consists of a one-story brick building, 117 ft. wide and 200 ft. long, covering 8 tracks, and capable of housing 24 locomotives. The roof, consisting of wooden trusses, is covered with slate, and contains 24 Roe cast-iron smoke jacks, which have since been connected up to 3 smoke stacks, 125 ft. high, with electric fans at the bottom. Beneath the track there is an ash-pit lined with concrete, and transversely to the length of the building there is a pit for engine repairs. The drainage is into the low-level sewer on the line of Twentieth St. Steam heat, sufficient to heat the building to 50° Fahr. in the coldest winter weather, is provided from the boilers in the repair shop.

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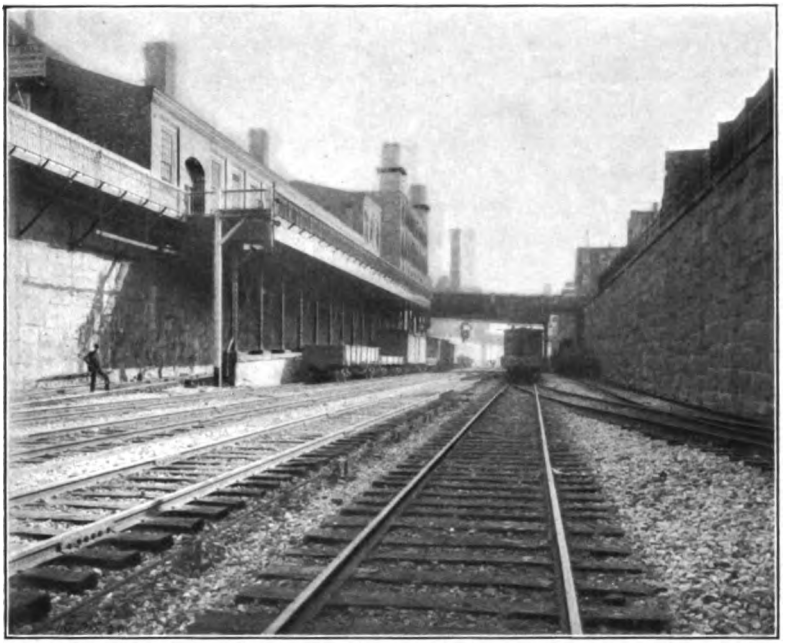

FIG. 1.—OVERHANGING DRIVEWAY AT EIGHTEENTH STREET.

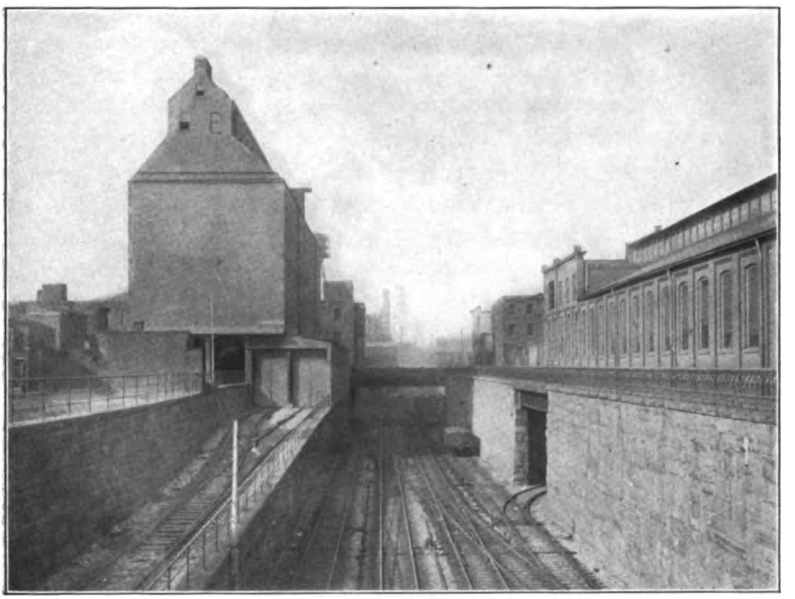

FIG. 2.—GRAIN ELEVATOR AND CONNECTION AT NINETEENTH STREET.

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The repair shop consists of a one-story brick building located in the corner of the depressed yard at Twentieth and Hamilton Sts., the retaining walls on the above-named streets forming two of the walls. It is 42 ft. wide and 82 ft. long, and contains two boilers, each of 175 H.-P., with two Argand steam blowers to burn buckwheat or rice anthracite coal. The boilers are guaranteed for a working pressure of 150 lbs. per square inch. Two duplex, boiler feed-pumps 5½ x 3⅛ x 5 ins., are provided, together with a water-heater. Two 100 K.-W., compound-wound, multipolar, direct-current dynamos of 220 volts each, with a speed of not more than 700 revolutions per minute, are provided, each with a high-speed engine, and the whole plant is furnished with the necessary switch-board and all other appliances. These dynamos and engines are for the purpose of operating the 50-ton electric crane for raising heavy freight to the street level. The prices bid for the work were as follows:

Engine-house and repair shop	$40 570
Boilers	11 255
Engines and dynamos	11 000

During the construction of the work, certain changes in the arrangement of the buildings were asked for by the railway company, entailing an additional cost of $4 756. The total cost of the work on the freight-house, engine-house and repair shop, as described above, was $141 999.80.

The contractor began work on April 25th, 1899, and finished on May 25th, 1900.

Proposed Power Lifts.

The power lifts, advertised for under Contracts 20 and 40, deserve more than a passing word of notice, on account of their size as well as of the engineering difficulties presented, and, although they were never constructed, the writers feel that their experience, as far as it went, may prove of interest.

In both contracts, simply the problem and the governing conditions were specified, and no design was made by the engineering force.

At one time two lifts were proposed, one at Seventeenth St. and the other at Nineteenth St., for the purposes already stated.

On account of the vigorous protest from the Philadelphia Grain Elevator Company, the lift at Nineteenth St. was abandoned, and the

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connection was made by alterations in the grain elevator, and by a track reaching the plant on a 4% grade.

The specifications for the lift at Seventeenth St., under Contract 20, provided as follows:

"The hydraulic lift is to be located on the south side of Pennsylvania Ave., east of the Seventeenth St. bridge. Its object is to lift an engine and three cars from the level of the subway to the present level of Pennsylvania Ave. (about 26 ft.). It is to be designed, constructed, and erected complete, including excavations, foundations, structural steel work, machinery, pumps, pipes, and all appurtenances (except the ties and rails for the track) needed and required to make the plant complete and ready for operation.

"The contractor shall submit with his proposal complete drawings and specifications covering the general features described herein, which drawings and specifications, when accepted by the Director of the Department of Public Works, shall constitute an essential part of this contract, and shall be binding severally upon the contractor and the City of Philadelphia."

The lift was to have been 152 ft. in length, with a rise of 26 ft. 2 ins. and with a distributed load of 420 000 lbs., as a maximum.

The general specifications further provided that:

"It shall be designed so that it can lift all or any part of this loading in any position.

"In addition to the general plant shown on the plans, the contractor shall provide and include in his price a pumping plant or other machinery for operating the lift. Space will be provided for these pumps in the power-house between Eighteenth and Nineteenth Sts."

As a result of the advertisement of May 12th, 1896, six bids were received for the hydraulic lift (Contract 20). The prices ranged from $48 500 to $138 825 for the whole work erected complete. One bid was for an electric lift, at $59 970. No two of the plans submitted were upon exactly the same principle, the number of rams or screws varying from one to four.

The merits of all these lifts were examined carefully. As a result, all the bids were thrown out, and a new specification was prepared, under Contract 40, for a power lift. The general dimensions and loading were not changed. The essential features specified, as relating to the design, were as follows:

"In addition to the lift, the contractor shall provide and include in his proposal a complete plant of the machinery and apparatus necessary to operate the lift. The machinery generating power shall be in

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duplicate. Space will be provided for the machinery in the power-house, to be located as shown on the general plan, on the south side of Pennsylvania Ave., west of Eighteenth St., on or about the present level of the street, and about 750 ft. west of the location of the lift.

"The city will provide boilers and their accompaniments, but the contractor for the lift shall provide and connect steam pipes to the main steam pipe in the boiler-room; also exhaust pipes to the feed-water heater, and main exhaust and drips from the pipes and the machinery operating the lift. He shall also provide covering for said pipes.

"A satisfactory means shall be furnished and constructed for conveying the power from said machinery in the power-house to the lift, and also a system of signals between the power-house and the lift to regulate the operation of the same.

"The contractor shall put the entire plant into operation, and supply everything for its proper service with the exception of the boilers and accompaniments.

"The following requirements shall be provided in all designs submitted, and the general methods for accomplishing the same shall be specified:

"*First.*—Absolutely uniform motion in the ascent and descent of the lift shall be maintained, and the platform kept horizontal under all conditions of loading.

"*Second.*—In case of accident of any kind, the platform shall come to a full stop in a horizontal position.

"*Third.*—Suitable protection shall be provided for the working parts of the lift, to prevent damage and wear from exposure to the weather, dirt, grit, water, etc.

"*Fourth.*—A method of operating the lift which can be controlled at the site will be preferred, all other things being equal.

"*Fifth.*—The lift, carrying the load under any condition, shall be capable of being raised or lowered throughout its entire vertical height in ten minutes.

"*Sixth.*—Special attention should be given to the design of the lateral stiffness of the lift, and the cost of making any attachments for the same to the masonry wall shall be included in the price bid.

"The preliminary drawing shall show such attachments, and any additions to the present masonry, if required.

"Should hydraulic power be proposed to be used, special attention is called to the distance of the power-house from the lift, and provision shall be made for the effects of the compression of the liquid used under the high pressures necessarily employed."

The work was again advertised, and, on March 30th, 1897, two bids were received for a hydraulic lift, $95 425 and $77 000, respectively, and two for an electric lift, $44 750 and $75 110.

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An examination of the designs submitted showed them to be practically free from objectionable features, but all bids were again rejected on account of the purchase of the Whitney property by the railway company and the substitution of a 5½% grade and a switchback in its place.

During the study of this matter, Table No. 15, giving a list of the then existing lifts, with their heights, pressures and loads, was prepared. Hydraulic power is used in all.

TABLE No. 15.

Name of lift and location.	Height, in feet.	Pressure.	Load, in tons.
Glasgow Harbor Tunnel	72	750 lbs.	6
Eiffel Tower	420	.....	3.5
Weehawken	143	.....	10.5
Mersey Tunnel	100	660 lbs.	7
Anderton, England, Canal	50	554 lbs.	510
Canal due Centre, Belgium	55.5	.....	.....
Les Fontinettes, France	43	.....	300
Proposed Subway	96.25	2 000 lbs.	210

Fifty-Ton Electric Crane.

On the south side of Hamilton St., just east of Twenty-first St., there is located a 50-ton electric traveling crane, which was built under Contract 27 with E. D. Smith and Company.

The plant consists of two lines of runway girders supported upon columns, the total length of runway being 96 ft. One runway is located 14.6 ft. north of the retaining wall of the subway, on piers built to the elevation of Hamilton St. The other is situated 29.2 ft. south of the same wall, the tops of the piers being at the level of the tracks in the depressed yard and the bridge of the crane spanning two of the tracks. The crane is carried by two trucks on each runway, which are 15 ft. between centers. The distance from center to center of the runway girders is 43 ft. 10 ins. The crane was built by William Sellers and Company, Inc., and was designed upon the following data:

Capacity, 50 tons;
Maximum lateral speed of crane, 100 ft. per minute;
To hoist the maximum load to the full height in 4 minutes;
To raise the maximum load as frequently as may be desired.

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The electrical cable connections with the dynamo were included under Contract 25. The distance from the subway tracks to the hook of the crane is about 56 ft.

The contract price for the crane, complete, including foundations, runway girders and all appurtenances, was $20 640.

Approximate Total Quantities.

The following list, of the total quantities of the more important items of the work, is given for the purpose of comparing the magnitude of the work with other work of a similar character:

Earth and rock excavation	1 086 422 cu. yds.
Masonry (exclusive of sewers)	183 114 " "
Temporary track	11.28 miles of single track.
Permanent track	13.45 " " " "
Sewers	7.02 miles.
Bridges	18
Structural steel in bridges, etc.	10 515 787 lbs.
Number of approved drawings, not counting studies	1 354
Number of shop drawings, checked and approved	1 010

Consequential Damages and Financial Summary.

Under the terms of the Ordinance appropriating the $6 000 000 for the work, and authorizing its construction, it was provided that all the expenses incident to its completion, including consequential damages and the widening of Pennsylvania Ave. from Twenty-second to Green Sts. and from Green St. to Twenty-sixth St., were to be paid out of the appropriation.

Juries of View appointed by the Courts, in accordance with Acts of Assembly, were constantly engaged in hearing claims of property owners and tenants, numbering in all about 182 claims. The claims made and filed by these owners and tenants amounted to approximately $2 000 000.

Wherever it was possible, no claims for change of grade were considered by the Juries until the physical changes were completed, so that there could be no speculation as to the exact nature of the damage. On this account, various sections of the work were considered as the work of construction advanced, and awards were made in advance of other sections where the work was still in progress.

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The following is a concise statement of all expenditures made on account of the work:

Total amount paid for contracts (including sewers*)	$4 067 482.15
Paid by mandamus for land and consequential damages	1 040 345.71
Fees for expert testimony in damage cases	29 185.05
Fees of Road Jurors in damage cases	3 826.00
Labor—Water, Gas and Electrical Bureaus	41 102.00
Engineering, inspection and incidentals	227 422.51
Total expenditures to December 31st, 1901	$5 409 363.42
Balance on hand December 31st, 1901	15 636.58
Total amount of loan negotiated	$5 425 000.00
Amount of loan not sold	575 000.00
Total appropriation	$6 000 000.00

Summary of Construction Dates.

The work of constructing the sewers was begun on September 10th, 1894, and by July 1st, 1895, the main sewers were completed. No construction work was then done until August, 1896, when the temporary track work was started. The first regular passenger train, carrying the members of the American Society of Civil Engineers on their way home from the Annual Convention at Cape May, passed through the subway on July 1st, 1899, and on September 25th of the same year the regular traffic of the railway was begun. By November, 1900, the work was practically completed, except a few minor details.

Engineering.

The ordinance authorizing the work provided that it was to be organized and carried out by the Director of the Department of Public Works of the City of Philadelphia, who has supervision of the Engineering Bureaus of that city. The Directors under whom the work was carried out were as follows: From September, 1894, to April, 1895, Hon. James H. Windrim, under Hon. Edwin S. Stuart, as Mayor. From April, 1895, to April, 1899, Hon. Thomas M. Thompson, under Hon. Chas. F. Warwick, as Mayor. From April, 1899, until the completion of the work, Hon. Wm. C. Haddock, under Hon. Samuel H. Ashbridge, as Mayor.

* For sewers, see *Transactions*, Am. Soc. C. E., Vol. xliv, page 1 *et seq.*

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PLATE XXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLVIII, No. 930.
WEBSTER AND WAGNER
ON PENNSYLVANIA AVENUE SUBWAY.

FIG. 1.—SUBWAY, EAST FROM SIXTEENTH STREET.

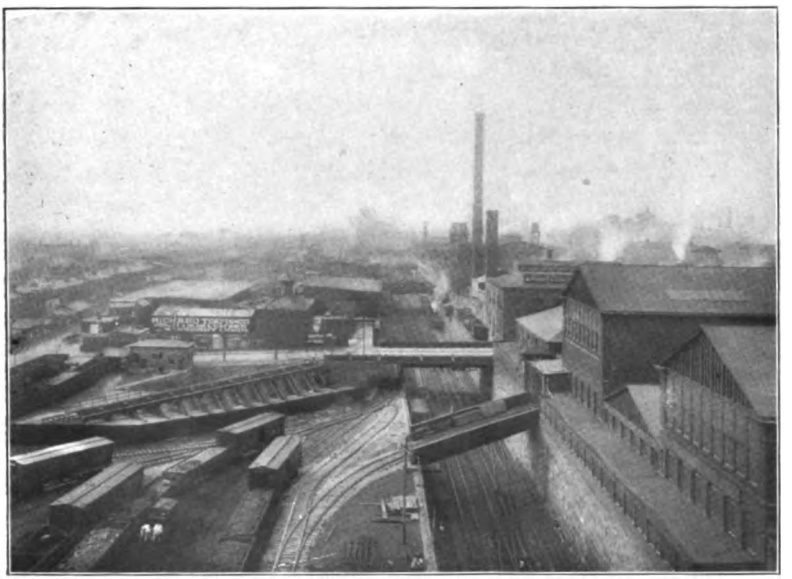

FIG. 2.—SUBWAY, WEST FROM SIXTEENTH STREET.

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The entire charge of the preparation of the plans and the construction was placed under George S. Webster, M. Am. Soc. C. E., Chief Engineer, Bureau of Surveys, which Bureau has jurisdiction over such work. Mr. George E. Datesman is Principal Assistant Engineer of this Bureau.

In June, 1894, Samuel Tobias Wagner, M. Am. Soc. C. E., was appointed First Assistant Engineer in charge of the work, and Mr. Charles H. Swan, Chief Draughtsman.

In August, 1894, Mr. Richard I. D. Ashbridge was appointed Second Assistant Engineer in charge of the construction of the sewers.

When active work on the main operations was begun in August, 1896, the field corps was organized with D. Jones Lucas, M. Am. Soc. C. E., as Second Assistant Engineer in charge of construction, and Mr. R. I. D. Ashbridge as Third Assistant Engineer in charge of the lines and grades. The principal Assistants under Mr. Ashbridge during construction were Mr. Frank R. Fisher, Mr. John W. Weaver, T. N. Spencer, Assoc. M. Am. Soc. C. E., and Mr. Stephen Harris. To all of these gentlemen great credit is due for skill and accuracy.

Under the Ordinance, the Director of the Department of Public Works is directed to confer, during the progress of the work, with an engineer appointed by the Philadelphia and Reading Railroad Company in regard to the approval of all plans and the proper execution of the contracts. The late John A. Wilson, M. Am. Soc. C. E., served in the capacity of Consulting Engineer from March 30th, 1894, until his death on January 19th, 1896. Mr. Wilson was closely identified with the preparation of all the detailed studies of the work previous to the passage of the Ordinance. Upon the death of Mr. Wilson, Mr. Joseph M. Wilson, M. Am. Soc. C. E., was appointed Consulting Engineer, and served until the completion of the work.

During the entire time, the principal officers of the Philadelphia and Reading Railroad Company were: Mr. Joseph S. Harris, President; Theodore Voorhees, M. Am. Soc. C. E., First Vice-President; Mr. H. K. Nichols, Chief Engineer, and William Hunter, M. Am. Soc. C. E., Assistant Chief Engineer.

To all the above-mentioned gentlemen the writers desire to acknowledge the kind assistance, advice and courtesy at all times rendered during the preparation of the plans and the carrying out of all the details of the work.

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				Date of proposal.						
8	Tearing down old houses at Pennsylvania Ave. and Pagoda St.....	Geo. W. Ruch.....	Mar. 18, '95	Mar. 18, '95	Mar. 27, '95	30 days	Apr. 72, '95	400.00	400.00	Apr. 9, '95
9	Removing surplus material on Wood St., and paving and repaving sidewalks.....	This contract was	never exec	uted. The w	ork was do	ne by C. P.	Grimf & Co.	as part of	Contract No.	1.
10	Tearing down old houses and stable at No. 2427 Pennsylvania Ave.....	Geo. W. Ruch.....	Oct. 28, '95	Oct. 28, '95	Oct. 31. '95 Verbal order.	10 days	Nov. 10' 95	85.00	85.00	Nov. 12, '95
11	Temporary tracks.....	P. McManus.....	May 12, '96	Aug. 3, '96	Aug. 10, '96	3 months	Nov. 3, '96	150 000.00	144 028.99	July 12, '98
12	Work from south side of Callowhill St. to the west house line of 13th St., including changes of grade of 13th and adjacent streets.....	P. H. Flynn & Co..	May 12, '96	Sep. 3, '96	Oct. 1, '97	18 months	Apr. 1, '99	225 000.00	152 442.94	Nov. 28, '99
13	Construction of retaining walls and abutments on the north side of Callowhill St., between 13th St. and a point near 16th St., and on the south side of Noble St., from 13th to Broad Sts.....	E. D. Smith & Co..	May 12, '96	Sep. 4, '96	Sep. 23, '96	18 months	Mar. 23, '98	85 000 00	84 995.66	May 17, '98
14	Excavation of core from Sta. 10+9 to Sta. 51, including construction of bridges, grading and repaving of approaches, etc.....	E. D. Smith & Co..	May 12, '96	Sep. 4, '96	Apr. 15, '97	18 months	Oct. 15, '98	765 000.00	736 088 92	Oct. 5, '99
15	Construction of freight-houses, between 13th and Broad Sts.....	Ryan & Kelley....	May 12, '96	Apr. 26, '97	May 16, '98	9 months	Feb. 16, '99	113 000.00	108 768 23	Oct. 16, '99
16	Construction of retaining walls, and reconstruction of Block "A" on the north side of Pennsylvania Ave., between Broad and 15th Sts.....	M. McManus.....	May 12, '96	Sep. 10, 96	Apr. 1, '97	3 months	July 1, '97	28 000.00	24 295.21	Dec. 31, '97

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TABLE No. 16.—List of Contracts, Pennsylvania Ave. Subway and Tunnel, according to Ordinance of March 17th, 1894.

No.	Description of contract.	Name of contractor.	Date of letting.	Date of contract.	Notice to begin work.	Time limit.	Time expires.	Limit of contract.	Total amount paid.	Final payment.
1	Sewer on Wood St., 24th St. and Powelton Ave., and on Callowhill St. from the Schuylkill River to Sta. 19 with appurtenant sewers on 22d St., Hamilton St. and Pennsylvania Ave.....	C. P. Grim & Co...	Aug. 30, '94	Sep. 4, '94	Sep. 10, '94	4 months	Jan. 10, '95	$94 000.00	$80 342.75	Aug. 19, '96
2	Sewer on Callowhill St., from Sta. 19 to Sta. 38, with appurtenant sewers on 17th, 18th, 19th and 20th Sts.....	Geo. W. Ruch.....	Aug. 30, '94	Sep. 5, '94	Sep. 10. '94	4 months	Jan. 10, '95	75 000.00	8 705.13 60 957.58	Mandamus Dec. 4, '97 Jan. 26, '97
3	Sewer on Callowhill St., from Sta. 38 to 13th St., and on 13th St., from Carlton St. to Buttonwood St., with appurtenant sewers on Callowhill, 15th and 16th Sts.....	J. McCann.....	Aug. 30, '94	Sep. 4, '94	Sep. 10, '94	4 months	Jan. 10, '95	47 000 00	35 976.81	June 26, '95
4	Sewer on Powelton Ave., 24th St. and Pennsylvania Ave., from the Schuylkill River to Sta. 19 + 88.....	Ryan & Kelley.....	Aug. 30, '94	Sep. 4, '94	Sep. 10, '94	4 months	Jan. 10, '95	102 000.00	92 586.42	June 29, '95
5	Sewer on Pennsylvania Ave., from Sta. 19 + 88 to 30th St.....	J. A. Mundy & Bro.	Aug. 30, '94	Sep. 4, '94	Sep. 10, '94	4 months	Jan. 10, '95	138 000.00	126 496.15	May 31, '95
6	Sewer on 12th St., Buttonwood St., and on 13th St., from Whitehall St. to Carlton St.....	J. McCann.....	Aug. 30. '94	Sep. 4, '94	Sep. 10, '94	4 months	Jan. 10, '95	25 000.00	22 823.49	Jan. 29, '95
7	Test pits on Pennsylvania Ave.....	J. McCann.....	Jan. 28, '95	Feb. 12, '95	Feb. 16, '95	10 days	Feb. 26, '95	475.00	174.77	Oct. 23, '95

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	Freight, engine, and repair houses in Twentieth St. yard	P. McManus.	May 12, '96	Aug. 11, '97	Apr. 24, '99	10 months	Feb. 24,1900	142 000.00	141 999.86	
26	Locomotive coaling station in Twentieth St. yard	Link Belt Eng. Co.	May 12, '96	Oct. 17, '96	July 12, '99	7 months	Feb. 12.1900	60 000.00	59 458.13	Nov.14, 1900
27	Electric crane in Twentieth St. yard	E. D. Smith & Co..	May 12, '96	Dec. 30, '96	Nov. 19, '98	24 months	Nov. 19,1900	22 000.00	20 640.00	Nov. 28, '99
28	Construction of retaining walls, and reconstruction of Block "Q" on south side of Pennsylvania Ave., between 21st and 22d Sts	E. D. Smith & Co..	May 12, '96	Sep. 4, '96	Sep. 23, '96	18 months	Mar. 23, '98	17 000.00	16 284.20	Apr. 1, '99
29	Work from Sta. 51 to Sta. 100, including tunnel and ventilation conduits, etc., but not fan stations. A final contract, except permanent tracks	This contract was	not awarde	d, on account	of change	of plans. R	e-advertise	d as No. 33.		
30	Permanent tracks, including tearing up of temporary tracks, paving Hamilton St., engine turn-table	Ryan & Kelley	May 12, '96	May 3, '97	June 1, '98	4 months	Oct. 1, '98	260 000.00	238 036.47	Sep. 9, 1900
31	Signal plant	E. D. Smith & Co..	May 12, '96	July 19, '98	Aug. 11, '98	18 months	Feb. 11,1900	90 000.00	86 376.00	July 5, 1900
32	Tearing down houses on Pennsylvania Ave., between Pagoda St. and coal yard formerly occupied by Garber & Son	Geo. W. Ruch	Mar. 9, '96	Mar. 9, '96	Mar. 11, '96	5 days	Mar. 16, '96	99.50	99.50	Mar. 16, '96
33	Work from Sta. 51 to Sta. 100, including tunnel and subway. A final contract, except permanent tracks	E. D. Smith & Co..	Sep. 1, '96	Oct. 3, '99 Feb. 9, '97	Mar. 6, '97	18 months	Sep. 8, '98	15 000.00 1 000 000.00	1 014 992.97	June 5, 1900
34	Commercial coal conveyor	This contract was	never prep	ared. Contr	act No. 49	substituted	therefor.			
35	Water pipe	McNeal Pipe and Foundry Co	Oct. 27, '96	Nov. 24, '96		30 days		15 000.00	14 968.70	Mar. 21, '99
36	Lead	Wm. J. Ashmead..	Oct. 27, '96	Dec. 10, '96		As wanted		2 000.00	1 363.85	Oct. 19, '97
37	Gas pipe	McNeal Pipe and Foundry Co	Oct. 27, '96	Nov. 24, '96		30 days		6 500.00	1 437.33	July 5, '99

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TABLE No. 16—(*Continued*).

No.	Description of contract.	Name of contractor.	Date of letting.	Date of contract.	Notice to begin work.	Time limit.	Time expires.	Limit of contract.	Total amount paid.	Final payment.
17	Construction of retaining walls, and reconstruction of Blocks "B" and "K," Pennsylvania Ave., between 15th and 16th Sts.	P. H. Flynn & Co.	May 12, '96	Sep. 8, '96	Mar. 29, '97	18 months	Sep. 29, '98	103 000 00	100 741.82	Oct. 21, '98
18	Commercial coal conveyor	This contract was	not awarde	d. Design n	ot satisfact	ory to the	P. & R. R.	R. Re-adve	rtised as No.	34.
19	Construction of retaining walls, and reconstruction of Blocks "C" and "L" on Pennsylvania Ave., between 16th and 17th Sts.	P. H. Flynn & Co.	May 12, '96	Sep. 8, '96	Sep. 23, '96	18 months	Mar. 23, '98	138 000.00	108 018.49	July 13, '99
20	Hydraulic lift	This contract was	not awarde	d, on account	of change	of plans. R	e-advertise	d as No. 40	.....	.....
21	Construction of retaining walls, and reconstruction of Blocks "D" and "M" on Pennsylvania Ave., between 17th and 18th Sts.	P. H. Flynn & Co.	May 12, '96	Sep. 8, '96	Sep. 23, '96	18 months	Mar. 23, '98	115 000.00	105 557.70	June 10, '98
22	Construction of retaining walls, and reconstruction of Blocks "E" and "N" on Pennsylvania Ave., between 18th and 19th Sts.	P. H. Flynn & Co.	May 12, '96	Sep. 8, '96	Sep. 23, '96	18 months	Mar. 23, '98	88 000.00	72 102.12	Law case.
23	Construction of retaining walls, and reconstruction of Blocks "F" and "O" on Pennsylvania Ave., between 19th and 20th Sts.	E. D. Smith & Co.	May 12, '96	Sep. 4, '96	Sep. 23, '96	18 months	Mar. 23, '98	82 000.00	68 812.63	Law case.
24	Construction of retaining walls, and reconstruction of Blocks "P," "G" and "H" on Pennsylvania Ave., between 20th and 21st Sts.	P. H. Flynn & Co.	May 12, '96	Sep. 8, '96	Sep. 23, '96	18 months	Mar. 23, '98	146 000.00	145 782.61	Feb. 9, 1900

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55	Permanent telegraph line	The Standard Underground Cable Co.....	May 3, '99	June 8, '99	June 20, '99	3 months	Sep. 20, '99	7 500.00	5 088.68	Oct. 31, '99
56	Paving, repaving, curb setting and appurtenant work on Hamilton and adjacent streets,...	Michael O'Rourke.	Sep. 19, '99	Nov. 1, '99	Nov. 18, '99	4 months	Mar.18, 1900	45 000.00	36 541.70	Aug. 10,1900
57	Protection and painting of bridges and other work.....	Ryan & Kelley....	Mar.18,1900	Mar.21, 1900	May 15,1900	5 months	Oct. 15, 1900	32 000.00	27 517.95	Dec. 26, 1900
58	Underpinning and adjustment to building.	Hoopes & Townsend...	John Mitchell.....	Dec. 18, '99	Dec. 23, '99	Mar. 8, 1900	60 days	May 17, 1900	4 000.00	2 345.44	June19,1900
59	Electrical, cast-iron manhole frames and covers.	The Greger Mfg. Co.....	Dec. 19, '98	Oct. 11, '99	.....	As wanted	.....	1 200.00	991.49	Nov. 28, '99
60	Painting upper deck, 16th St. railroad bridge.....	J. W. Webber.....	Oct. 14, '99	No contract.	Oct. 20, '99	7 days	Oct. 27, '99	50.00	50.00	Dec. 5, '99
61	Automatic safety gates for elevators in 20th St. freight-house.....	Ryan & Kelley....	Oct. 2, 1900	Oct. 26, 1900	Jan. 5, '01	60 days	Mar. 5, '01	2 000.00	1 489.00	Apr. 26, '01
								Total	$4 067 482.15	

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TABLE No. 16—(*Concluded*).

No.	Description of contract.	Name of contractor.	Date of letting.	Date of contract.	Notice to begin work.	Time limit.	Time expires.	Limit of contract.	Total amount paid.	Final payment.
38	Telegraph cable	Western Electric Co	Oct. 27, '96	Nov. 27, '96	Dec. 9, '96	30 days	Jan. 8, '97	6 500.00	4 468.24	Mar. 23, '97
39	Gasket, jute packing, coke, etc	J. F. Hazard & Co., Inc. Uhler and English	Nov. 11 '96	No contract.		As wanted			English 28.33 Hazard 35.96	Apr. 23, '97 Apr. 7, '97
40	Power lift		Mar. 30, 97	This contract	not awarde	d. Lift ab	andoned.			
41	Permanent track connection, Block "F"	E. D. Smith & Co.	Aug. 24, '97	Sep. 1, '97	Sep. 13, '97	4 months	Jan. 13, '98	24 000.00	19 904.20	May 17, '98
42	Trestle and temporary tracks	E. D. Smith & Co.	Aug. 24, '97	Sep. 1, '97	Sep. 15, '97	2 months	Nov. 15, '97	29 000.00	19 049.18	Nov. 1, '98
43	Cast-iron gas pipe, valves and fittings	Camden Iron Works, Frank W. Dilks & Co.	Nov. 1, '97	Dec. 16, '97 Dec. 4, '97	Dec. 22, '97 Dec. 15, '97	1 month	Jan. 22, '98 Jan. 15, '98	A 1 200.00 B 450.00	897.31 422.00	May 24, '98 Dec. 29, '97
44	Retaining walls, grading, and other work, Block "L"	E. D. Smith & Co.	Mar. 3, '98	Apr. 12, '98	Apr. 18, '98	4 months	Aug. 18, '98	18 000.00	13 444.48	Jan. 24, '99
45	Construction of the work appurtenant to the temporary track connection from Hamilton St. to the Baldwin Locomotive Works	E. D. Smith & Co.	Dec. 21, '97	Jan. 12, '98	Jan. 15, '98	30 days	Feb. 15, '98	8 500.00	6 075.40	Mar. 6, 1900
46	Bridge railings	The North Penn. Iron Co	Mar. 3, '98	May 3, '98	May 7, '98	90 days	Aug 4, '98	6 000.00	4 941.51	Sep. 22, '99
47	Galvanized pipe for electrical conduits	Whetstone & Co	Jan. 18, '98	Apr. 1, '98				1 600.00	1 539.83	Apr. 15, '98
48	Railroad yard	Gordon Chambers.	June 28, '98	July 30, '98	Aug. 10, '98	90 days	Nov. 25, '98	1 900.00	1 900.00	July 31, 1900
49	Coal yard, 15th and Callowhill Sts	Ryan & Kelley	Feb. 23, '99	Mar. 2, '99	Apr. 10, '99	4 months	Aug. 10, '99	24 000.00	19 401.61	Apr. 24, 1900
50	Permanent water supply	McParland & Butler	Sep. 6, '98	Sep. 29, '98	Nov. 7, '98	30 days	Dec. 7, '98	7 500.00	4 766.62	Oct. 24, '99
51	Guard Rails for permanent tracks	Wilkins & Kuemmerk	Sep. 6, '98	Oct. 1, '98	Nov. 4, '98	60 days	Jan. 17, '99	6 000.00	5 220.10	July 24, 1900
52	Coal hopper pit. Bement, Miles & Co	Stacy B. Opdyke, Jr	Feb. 23, '99	Mar. 9, '99	Apr. 22, '99	3 months	July 22, '99	6 000.00	4 031.69	Oct. 6, '99
53	Railroad bumpers	Ryan & Kelley	May 3, '99	May 26, '99	July 1, '99	4 months	Nov. 1, '99	14 000.00	11 551.00	May 9, 1900
54	Repaving and temporary curb		Dec. 13, '98							

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DISCUSSION.

Mr. Wilson. JOSEPH M. WILSON, M. Am. Soc. C. E. (by letter).—The problem of the improvement of the Pennsylvania Avenue line of the Philadelphia and Reading Railroad (now Railway), in Philadelphia, including the abolishment of grade crossings at the streets, particularly at Broad St., was one that occupied the attention of the Reading Company and the City for years, becoming more or less involved in the general scheme for the new Reading Terminal Station and the abatement of grade crossings on the Ninth St. line.

The station was constructed and the Ninth St. line revised and rebuilt with overhead crossings as far as Green St.,* before any work was carried out on Pennsylvania Ave., by the late John A. Wilson, M. Am. Soc. C. E., who, as Chief Engineer in charge of the Terminal work, took an active interest in the Pennsylvania Avenue problem from its very inception, giving it a close study, consulting with the railroad company, the city officials, the citizens' committee, property holders, and others in interest, and presenting several schemes for consideration, together with plans, profiles and estimates, which were discussed and modified from time to time, until a solution of the question, practical and satisfactory to all parties, was finally reached.

The City Ordinance to authorize the subway work was finally enacted and approved on March 17th, 1894, and on March 30th Mr. Wilson was officially appointed Engineer of the Philadelphia and Reading Railroad Company, enabling him to confer with the officers of the Department of Public Works of the City upon the proposed improvement, as the representative of that company.

The numerous connections which the line of the railroad had with industrial establishments, coal yards, etc., the operations of which were intimately involved with some of the most important business interests of the city, and which had to be preserved as vital parts of the respective properties, without which the Reading Railroad system would be practically useless, complicated the problem seriously. In arranging these connections, the necessary conferences with the property owners, and estimates of their business requirements, etc., involved an immense amount of personal work, for which Mr. Wilson was peculiarly fitted, and although the Ordinance of Councils provided that all work authorized should be done under the direction and supervision of the Director of the Department of Public Works, who was authorized and directed to appoint such assistant engineers and draughtsmen, etc., etc., as were necessary for the prompt and efficient execution of the work, yet the mass of this preliminary work, including sketch-plans, was done directly under Mr. Wilson's eye, and

* *Transactions*, Am. Soc. C. E., Vol. xxxiv, page 115.

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the subway as it is now completed remains as a monument to his ability as a railroad engineer. Mr. Wilson.

At the time of Mr. Wilson's death (January 19th, 1896), the general plans and some of the detailed drawings had been approved by the City and the Railroad Company, and most of the changes in the sewers had been made, but no actual construction had been commenced on the track work, although the programme had been arranged for the method of procedure, the continuous maintenance of traffic, etc., which was adhered to throughout, almost literally.

The writer was appointed Consulting Engineer for the Railway Company after his brother's death. His duties involved the examination and approval of all drawings and specifications, and personal examination and expert advice on questions of carrying on details of the work. A few special problems remained to be studied out, some of these being particularly interesting, and new ones came up during progress.

The work was carried out effectively and energetically, reflecting great credit on the Department of Public Works, the Chief Engineer of the city and his assistants, and also on the contractors. It has been so well described by the authors that little or nothing could be added to the paper in that respect, and it stands as one of the most reputable pieces of engineering work ever executed by a city corporation, not only as to excellence of construction, but also as to the cost having been kept within the estimates and the appropriation.

CHARLES G. DARRACH, M. Am. Soc. C. E.—During the early inception of the work under consideration, while the late John A. Wilson, M. Am. Soc. C. E., was the Consulting Engineer of the Railroad Company, the speaker was his professional partner, to whose lot fell the duty of preparing a scheme for ventilating the subway tunnel. The general proposition presented was to ventilate the tunnel in such a way that it could be used as a part of the terminal-yard, and at the same time allow Pennsylvania Avenue, overhead, to be used as a grand boulevard—one of the magnificent thoroughfares of the city. Mr. Darrach.

To this end, the speaker designed the ventilation scheme shown in Fig. 7.

After a long discussion by the engineer of the Mining Department of the Railroad Company and the engineers of the City and of the Railroad Company, the plans were approved, and proposals asked on the work for that portion, exclusive of the fan vaults, fans, and ventilating towers. It is stated in the paper that the construction of the airways involved an expenditure of about $200 000 more than the tunnel without the airways.

About that time, according to the speaker's recollection, a question arose between the railroad company and the city authorities as to who should bear the cost of the maintenance of the ventilating plant;

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Mr. Darrach

Fig. 15.

Mr. Darrach.

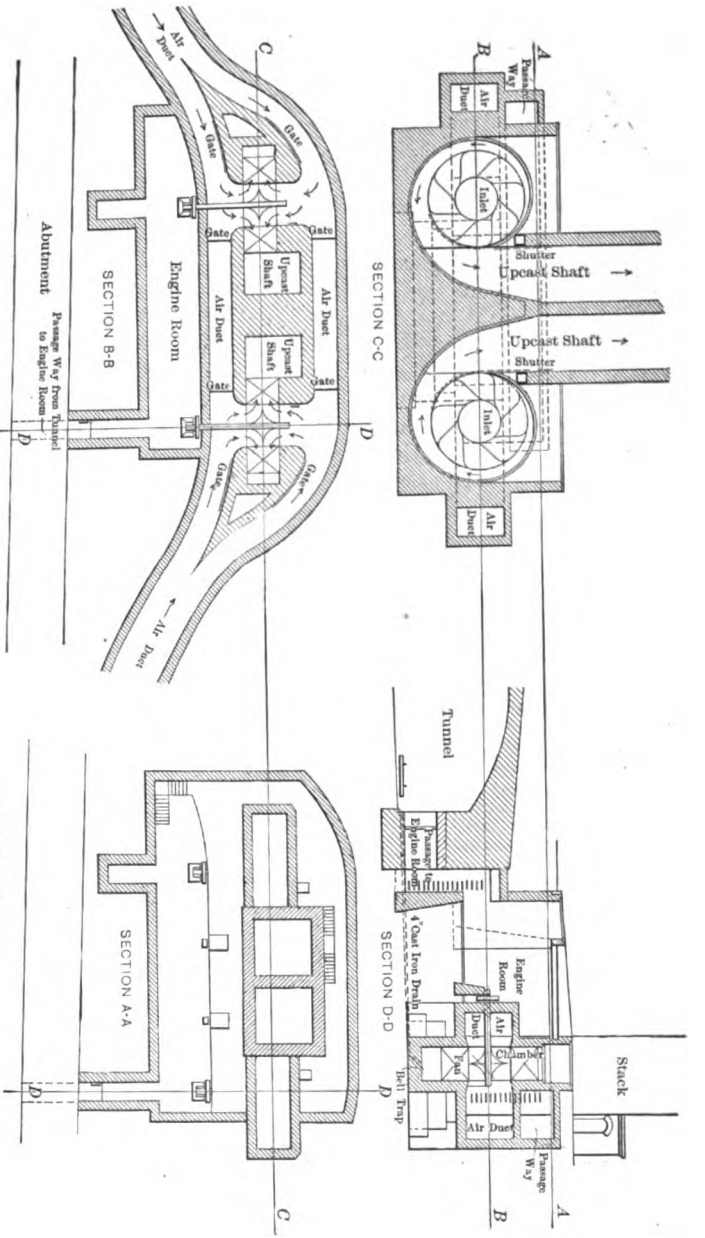

FIG. 16.

READING SUBWAY
SECTIONS OF FAN STATIONS

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Mr. Darrach. the city claiming that it should be borne by the railroad company, and, if the speaker remembers correctly, the railroad company maintaining that the cost should be divided or the railroad company credited with a capital which would represent the cost of one-half the maintenance. This question was compromised by going back to the method which was unqualifiedly condemned by all parties when the subject of the tunnel was originally discussed, and the result has been that neither the city nor the railroad company has obtained what they desired.

Pennsylvania Avenue over the tunnel is about 2 800 ft. in length, with a width of 80 ft. between building lines. It occupies one of the most valuable sections of the city, and if the original intention of the city had been carried out, the value of the property would have been increased materially; and not only the character of the street itself, but the entire character of that portion of the city would have been changed. It passes through what is known as Green Hill, immediately adjacent to Fairmount Park, on the brow of the steep slope, the elevation of which is 60 to 80 ft. above city datum, and, in the speaker's opinion, millions of dollars have been destroyed by what appears to him to have been shortsighted policy.

The difference in cost between the tunnel as constructed and the tunnel with airways was $200 000. If to this is added, say $200 000, for the completion of the work, the cost would have been, for interest at 3%, $12 000. The cost of maintenance and operation would have been approximately $6 000.

The form of construction adopted neither ventilates the tunnel properly nor does it make the overhead street a boulevard. During murky days the smoke and gases are decidedly in evidence through the middle of the street.

The method of ventilation suggested cannot be considered as absolutely original, as the speaker thinks a similar principle has been utilized in France, the idea being to ventilate across the tunnel from fresh-air inlets at the bottom of one side of the tunnel to foul-air outlets at the top of the opposite side. The fresh-air inlets were to have been perfectly free and without barriers, and the outlets were arranged with adjustable dampers so that the frictional resistance, impossible to determine by calculation, could have been adjusted by experiment.

The fans were to have been in duplicate, and operated either by steam, gas, gasoline, compressed air or electricity. The two fan stations were arranged as shown in Figs. 15 and 16. At each fan station a tower, Fig. 17, and Plate XXIII, was to have been erected, discharging foul gases, smoke and steam far above the housetops. The fans were to have been installed in duplicate at each station, to be operated either in part or in whole, as the necessities of ventilation required.

This subject is particularly interesting at a time when the tunnels

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Mr. Darrach.

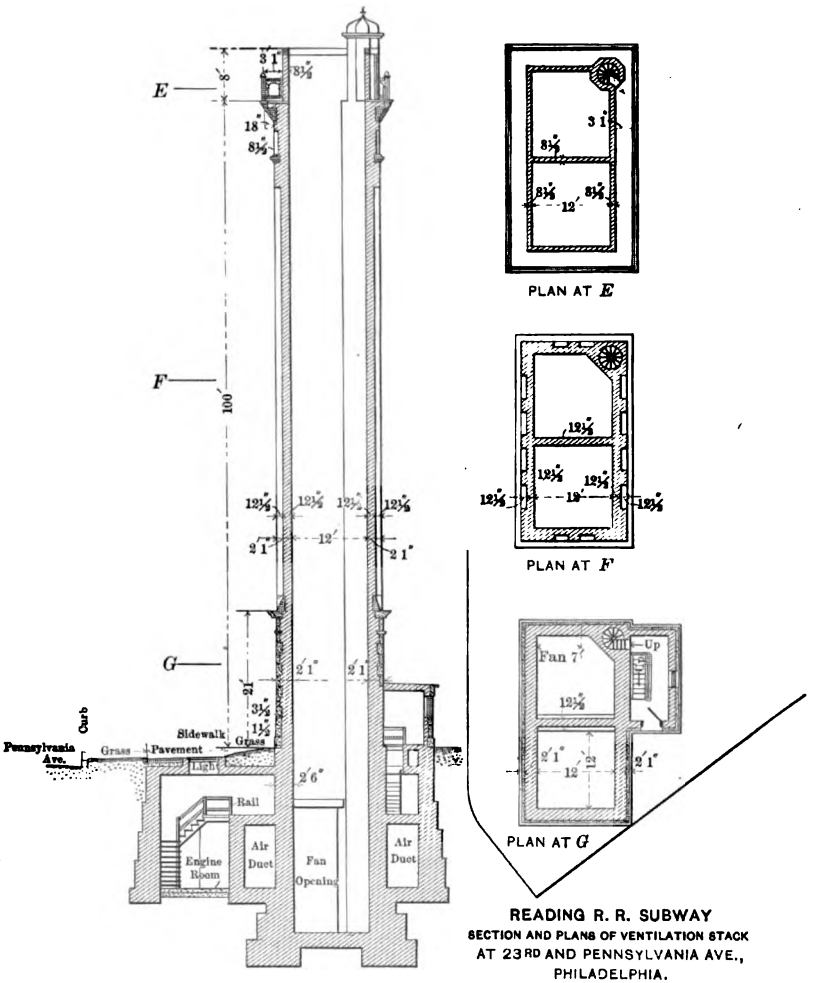

FIG 17.

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Mr. Darrach. under New York City and the adjacent rivers and bays are under consideration. The speaker is of the opinion that the principle involved is correct, for by its use, and special adaptation to local conditions, the trains would not have been enveloped in either their own smoke and gases or those of other trains.

Mr. Parsons. WILLIAM BARCLAY PARSONS, M. Am. Soc. C. E.—On account of the great development of subway construction in different cities, this paper is probably a forerunner of many other papers of a similar nature.

Subways are of two general types: those which are to be used by surface railroads, and those to be used by local inter-urban lines. The subway described in this paper is purely a subway of the former class, and as the railroad which is to make use of it is operated by steam locomotives, the standards used in its construction are those found in ordinary steam railroad construction, in addition to which a special provision had to be made for ventilation.

These conditions differ a good deal from the subway practice with which the speaker is more especially interested. The construction of the subway described is wholly of masonry, except so far as the overhead bridges are concerned, and the steel-girder construction around the ventilating openings. The walls supporting the arch and the overhead bridges, and the arch itself, are of either rubble or brick masonry, and it is to be noted that no attempt is made to utilize either concrete or a combination of steel and concrete.

In the work now in progress in New York City the latter form of construction has been used almost exclusively, and from an experience that has already been had, taken with the experience with steel in concrete on other similar work, the speaker is quite confident that in the future more attention will be given to the combined use of steel and concrete, possibly going to very much greater lengths than in the subway in New York, in using, not merely the steel beams embedded in concrete, but concrete, reinforced with expanded metal, or steel in some cheaper or lighter form than the ordinary steel beam.

The speaker has made some experiments with beams embedded in concrete and tested to destruction. Beams in pairs, carefully strutted so as to prevent failure by flexion, failed at a fiber stress of approximately 52 000 lbs. Tests taken from the flanges of these beams, and tested in a testing machine, showed that the material under those conditions had a tensile strength of 62 000 lbs. The lower figure of the large-sized test is probably explained by the fact that it is impossible to load the beams so as to distribute the load exactly evenly. Similar beams were then embedded in concrete, carrying the concrete 2 ins. above the top of the 6-in. beams and then loading them as before. Assuming that the concrete itself acted only to distribute the load

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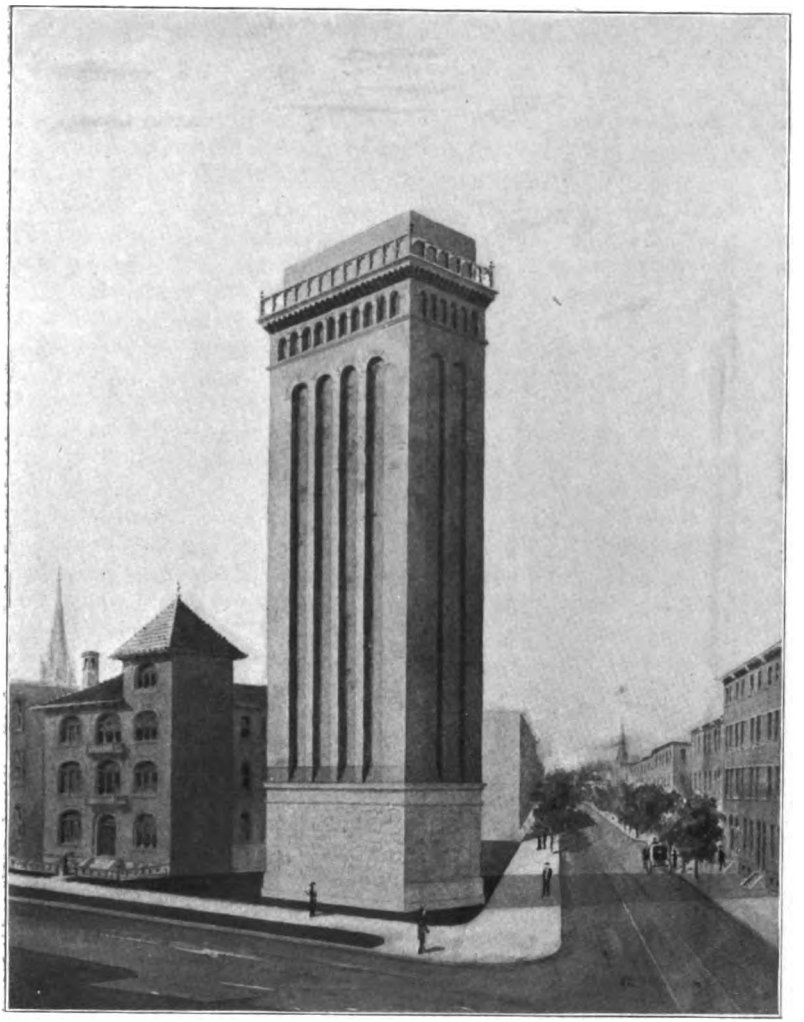

PROPOSED VENTILATION TOWER.

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and did not act as a beam at all, that is, assuming that the steel beams carried the whole of the load, the fiber stress at which the beams failed was 85 000 lbs. This figure indicates clearly that it is erroneous to compute the strength of the combined structure as the strength of the beams alone, but that the concrete, as well as the beams, takes its share of the load. Mr. Parsons.

It is to be hoped that, whatever discussion the paper may cause, attention will be paid to the radical difference in design that this subway presents as compared with the subway now being constructed in New York City, or that already constructed in Boston.

GEORGE A. SOPER, Assoc. M. Am. Soc. C. E.—(by letter).—The writer has been greatly interested in this valuable paper, and believes that in treating the subject so exhaustively the authors have performed a useful service. Particularly is this so in connection with the question of the attempt made to keep the air of the tunnel in an acceptably pure condition. Thanks to the thoroughness with which the work has been described, we know what was proposed in the way of ventilation and what was actually done. A comparison of the two plans teaches a very instructive lesson. Mr. Soper.

It was at first the intention of the railroad company to build a wholly covered subway. As the traffic was to be heavy, and the tunnel was located in a built-up portion of the city, the question of ventilation was more than ordinarily important and difficult. It was intended that the gases and smoke of the locomotives should be prevented, if possible, from becoming a nuisance to passengers traveling through the tunnel and to people residing in the neighboring streets.

No fault can be found with the plan first made to ventilate the tunnel. The situation called for exceptional measures. The most effective means known for ventilating tunnels were brought into requisition, and a highly elaborate system was designed. In view of the circumstances, there seems to be no reason to suppose that the plan was other than necessary and sufficient.

When the plan was completed and the cost of installing and operating the ventilating system was counted up, it seems that the Railroad Company was unwilling to stand the expense. In its view, it cost too much money to secure fresh air. An interview with the city authorities was held, and after fruitlessly trying to induce the city to share the cost, the plan of ventilation was abandoned.

At this juncture the plan of the subway was radically changed. Instead of a closed tunnel with a mechanical ventilating system, the road was to be converted into little more than an open cut. No artificial ventilation was to be used. Instead of stacks to carry the foul air to a point high above the ground, fans worked by electricity, automatic shutters to prevent gases escaping to the street, and fresh-air ducts opening upon grass plots, there was substituted no mechanical device or contrivance whatsoever.

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Mr. Soper. It is a very significant fact, in the writer's view, that this change of plan was made. It demonstrates in a very practical way that a closed city subway, for coal-burning locomotives, and with a heavy traffic, cannot be ventilated properly at a cost which is within the limit of the means of a large and wealthy corporation.

Considerable experience has been had in ventilating tunnels in various parts of the world, and the results show conclusively that there is no way by which the air of tunnels can be quickly and cheaply renewed. The air of few tunnels is even approximately satisfactory. The Mont Cenis Tunnel, the Hoosac Tunnel, the underground roads of Paris, Liverpool, Glasgow and Boston, and the St. Louis, Washington, Baltimore, Philadelphia and New York tunnels are imperfectly ventilated. The air is usually heavily charged with gases and smoke, and is at once uncomfortable to breathe and detrimental to health.

Practically, the only reliable method of ventilating tunnels requires the use of mechanical fans, and the operation of these involves continuous and considerable expense. There have been people who have thought that the piston action of moving cars was sufficient to secure a sufficient interchange of air; that foul gases, being warmer than the atmosphere, would readily escape if vertical shafts were provided to carry them off; that the currents produced by the rapid passage of trains would cause foul air to rush up and fresh air down through blow-holes, and that large stations or occasional openings would afford outlets for undesirable gases. Unfortunately, experience has shown that too much reliance has been placed upon these cheap and easy methods, and that, in the ventilation of large and important tunnels where coal is used, they are not based upon a correct interpretation of the laws of physics or sanitation.

Aside from the use of fans, the most beneficial results have followed the opening up of large portions of the tunnel roof to the outside air; but this is not so much solving as escaping the problem of tunnel ventilation. In proportion as a tunnel loses its covering, it becomes an open cut.

It may be interesting to note the ventilating arrangements of some well-known tunnels. At Paris, an attempt is made to ventilate the underground road of the Chemin de Fer de Sceaux by exhausting the impure air with fans at the highest point of the line, fresh air being allowed to enter at the stations and through shafts. Blow-holes are located from 200 to 300 ft. apart.

At Liverpool, the tunnel under the Mersey, 1.2 miles long, is provided with a ventilating equipment consisting of a 7-ft. ventilating conduit, alongside the tunnel, provided with fans. Openings communicate between the tunnel and the conduit at intervals. Foul air is forced out through this conduit and fresh air enters at the ends of the tunnel. Analysis has shown that the carbonic acid in the air of the Mersey Tunnel varies from 7.4 to 26.4 per 10 000 volumes.

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Mr. Soper

The air in the subway of the Metropolitan Railway of London has long been a source of complaint. A Government Commission, in 1897, found that the carbonic acid ran as high as 89.4 parts per 10 000, and it was recommended that the tunnel be opened as far as practicable by removing portions of the roof.

There is more in this problem of city subway ventilation than might appear at first sight. The construction of underground railroads, by which passengers from a distance can enter our great cities or pass from point to point within the city limits with convenience and immunity from danger to life, limb or health, is reaching enormous proportions. Not only have London, Paris, Berlin, Buda Pest Boston and New York their subways, but the number of other cities which are desirous of improving their railway communications in the same direction is legion.

The ventilation of city subways is a sanitary problem, and should be approached after the manner of solving sanitary problems in general. This does not always mean the removal of matters which are obnoxious, or injurious to health. Sanitation in its comprehensive scope often means prevention.

Applying this principle to the problem of ventilating the Pennsylvania Avenue Subway, it is seen at once that the surest way to have insured good air was to have substituted electric traction for coal-burning locomotives. Proper ventilation would then have been economically possible.

Mr. Humphrey.

RICHARD L. HUMPHREY, Assoc. M. Am. Soc. C. E. (by letter).—There are a few facts connected with the tests of the materials used in the construction of the Pennsylvania Avenue Subway and Tunnel which have not been brought out in the paper, and which may perhaps be of interest.

The specifications provided that the masonry should be laid in cement mortar, using three parts of bar sand where Portland cement was required, and two parts of bar sand where natural cement was required.

In the excavation of the core for a portion of the subway and tunnel a gravel was encountered, and the contractor applied for permission to substitute this for the bar sand required by the specifications.

The gravel, as excavated, contained a large percentage of coarse pebbles, ranging from $\frac{3}{4}$ in. to 2 ins. in diameter. The contractor, therefore, was required to screen out all pebbles larger than $\frac{1}{2}$ in. in diameter. Thus screened, it was found to be exceedingly well proportioned as to the size of the grains, and, although it contained fine particles (principally clay), to the extent of about 8%, it yielded excellent tests, both in compression and tension.

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Mr. Humphrey. The following is the average result of sieving a number of samples:

Residue on No.	10	sieve	.....	35.3	per cent.
" "	20	"	.....	32.1	"
" "	30	"	.....	14.6	"
" "	50	"	.....	9.6	"
" "	100	"	.....	4.9	"
" "	200	"	.....	2.0	"
Passing "	200	"	.....	1.5	"
				100.0	"

The gravel contained about 33% of voids, and was used in the proportion of three parts gravel to one part Portland cement.

The cement used in all portions of the subway, whether natural or Portland, was of the highest quality, as will be readily seen from an inspection of the average results of the tests, in Table No. 4.

As the screened gravel contained particles as large as ½ in. it could not be used in brick or ashlar masonry, where close joints were required; and, as the contractor did not care to go to the expense of screening out the coarse particles, the gravel could only be used in laying rubble masonry and in concrete.

The bar sand required by the specifications is that in general use in Philadelphia, and is dredged from the bars formed at certain points in the Delaware River by the action of the tide.

This sand is fairly clean, and consists of fine rounded particles, containing about 39% of voids, and leaving the following residues when screened:

Residue on No.	20	sieve	.....	12.8	per cent.
" "	30	"	.....	49.0	"
" "	50	"	.....	29.3	"
" "	100	"	.....	5.7	"
" "	200	"	.....	2.3	"
Passing "	200	"	.....	0.9	"
				100.0	"

In addition to the regular laboratory tests of the cement as it was delivered on the work, tests of the mortar and concrete were made.

Briquettes were made from the mortar and 6-in. cubes were made from the concrete taken from the mixing box. These test specimens were immersed in water after being exposed to the air from 24 to 48 hours.

In the construction of the subway it was necessary to rebuild portions of some of the buildings. In such cases the brickwork above

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Mr. Humphrey.

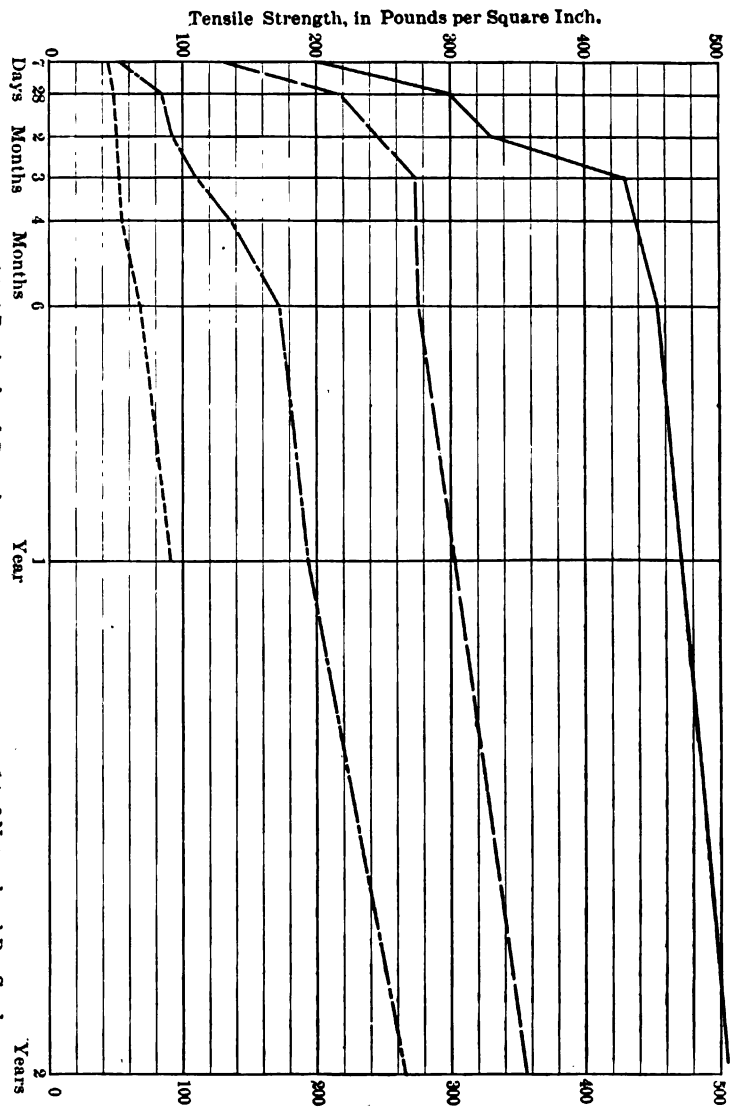

Fig. 18.

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Mr. Humphrey.

the foundations was laid in lime mortar. Briquettes were also made from this mortar taken from the mixing box.

It was found that this lime mortar hardened very slowly, that it was impracticable to remove the briquettes from the moulds for several days after moulding, and that they could not be placed in water, without dissolving and disintegrating, until they had been allowed to harden in the air for at least one month. The briquettes, therefore, were preserved in the air until broken.

Fig. 18 shows the average results of the tests of the briquettes made from the following mixtures (taken from the mixing box): Portland cement, gravel and bar sand; natural cement and bar sand; and lime and bar sand. Fig. 19 shows a comparison of the compressive strength of Portland, gravel and stone, and of Portland and gravel.

It is interesting to note what an inferior mortar lime makes; and the inferiority becomes much more marked when it is considered that these tests were made under far more favorable conditions than would be possible in actual work, *i. e.*, the mortar was protected from the action of the weather during hardening.

It is not surprising, therefore, that early constructors found lime mortar so deplorably bad, and that such mortar deteriorated so rapidly that the joints of masonry had to be repointed frequently.

Fig. 18 also shows the superiority of natural cement mortar over lime mortar, and the marked superiority of Portland cement mortar, containing 50% more sand, over the natural cement mortar.

While natural cement mortar acquires considerable hardness, it requires from 6 to 12 months and longer to develop any considerable strength. During this period it must be handled with care, and will not stand exposure to temperatures below freezing without disintegrating. It should also be borne in mind that the natural cement used in this work was from 50 to 100% stronger than the cement ordinarily found in the market; this will be apparent from an inspection of the tests in Table No. 4.

Portland cement, on the other hand, develops its strength much more rapidly, will carry a much greater proportion of sand for a given strength, and is not seriously affected by freezing after it has once set. When these facts are also taken into consideration, the superiority of Portland cement mortar becomes still more apparent.

Another interesting feature illustrated by this diagram is the improvement in the quality of the mortar used in construction.

The lowest line represents the mortar commonly used in the earliest constructions, the next line above shows the old Roman mortar and the best mortar available, up to 1824, while the third line shows the quality of the mortar now in general use, and which was made possible by the discovery of Aspdin, in 1824.

While the diagram shows the marked superiority of this latter

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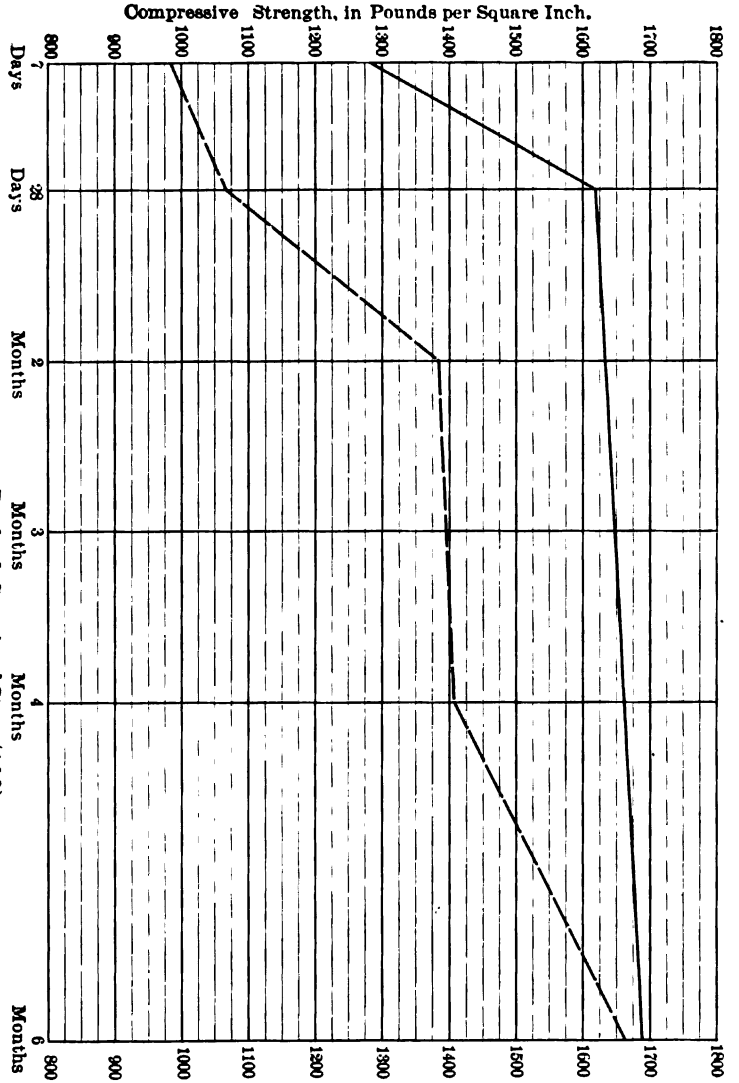

FIG. 19.

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mortar over the others, it does not show the important qualities, *viz.*, the rapidity of hardening, the capacity for sand, and the resistance to the action of the weather.

The superiority of the gravel over the bar sand mortar is also readily apparent.

The value of a gravel mortar depends largely on the quality of the gravel. This quality is dependent on the character and size of the grains, the nature and percentage of the fine particles, and the condition of the surface of the grains. If the fine portion is largely clay of a fine, flaky nature, it often forms a coating around the particles, which prevents a good bond with the cement and reduces the adhesive strength of the mortar. In many instances this fine portion is quite sharp and gritty. A gravel in use in Philadelphia, known as "Jersey gravel," is an example of this kind. This material is well proportioned, perfectly clean, and, as a rule, contains no extraneous matter, or clay.

The results of sieving it are as follows:

Residue on	No. 10	Sieve	16.56	per cent.
"	" 20	"	32.92	"
"	" 30	"	14.78	"
"	" 50	"	20.30	"
"	" 100	"	12.54	"
"	" 200	"	1.84	"
Passing	" 200	"	1.03	"
			99.97	"

In a gravel, the presence of clay, up to 6%, is not necessarily objectionable, provided it has not coated the particles of the gravel, and does not adhere so closely as to render a good bond with the cement impossible.

Unless the coarse particles are screened out, down to the No. 20 sieve, gravel is not adapted for any work other than rubble masonry or concrete, in which close joints are not necessary.

A clean gravel, composed of well-graded particles, and free from an excess of clay, is one of the best materials for concrete. Using 1½ to 2-in. broken stone and gravel (in which all particles coarser than ½ in. have been screened out), an almost ideal mixture suitable for general use, would consist of one part Portland cement, three parts gravel and six parts broken stone.

The advantages of a gravel concrete may be summed up as follows:

1. It is much more dense than ordinary concrete, which is composed of sand and stone passing a 2-in. ring, because the particles of which it is composed are graded better;

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2. The voids (because of this better grading) being much less, it requires less cement in order to produce a given strength;

3. The particles of gravel being rounded, admit of greater compaction, which can be obtained much more readily, and with less ramming, than with a concrete composed of angular stone;

4. It attains greater strength in a given length of time.

In places in the subway where it was necessary to cut into or tear out portions of the completed work, the gravel concrete was found to be so hard that in many cases it could be removed only by blasting. In general structure it resembled a hard conglomerate.

Excellent gravels which are often found in or near the excavations for important engineering work are passed unnoticed, and crushed stone (carefully screened to the 1½ or 2-in. size) used. The crushed stone is often not readily obtainable, and must be hauled from a distance at considerable expense. Under such circumstances, a gravel would not only make a better concrete, but would also be more economical. It is unfortunate that the advantages in the use of gravel for concrete are not better appreciated.

In order to determine and have a record of the quality of the stone used in the subway, a series of tests was made on 2-in. and 8-in. cubes.

Tests were made on two sizes of cubes because it was believed that a test which would be more representative of the quality of the stone could be obtained from large cubes.

As it was claimed that some of the stone used would stand 24 000 lbs. per square inch, it was decided that an 8-in. cube was the largest size that could be crushed with the facilities available; and, as most of the compressive tests of stone are made on 2-in. cubes, it was also decided to make a series of tests on cubes of this size, for purposes of comparison.

The tests consisted in the determination of absorption, specific gravity, weight per cubic foot, and strength in compression.

Absorption.—The test specimens were first dried for 24 hours, or to constant weight, at 212° Fahr. They were allowed to cool slowly, until they had acquired the temperature of the testing room (about 75° Fahr.), and weighed. They were then immersed in water at normal temperature (60 to 70° Fahr.) for 48 hours, after which they were removed (the excess of water on their surfaces being absorbed by means of blotting paper), and weighed. The difference between this weight and the dry weight in air gave the weight of the absorbed water, which, divided by the dry weight gave the percentage of absorption.

Specific Gravity.—This determination was made on the thoroughly saturated stone, and therefore immediately followed the test for absorption.

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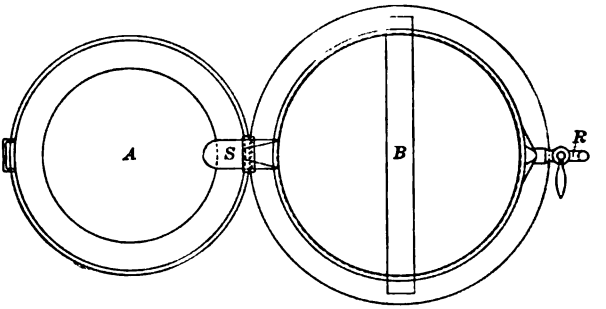
The smaller cubes (after removing with blotting paper the excess water on the surfaces) were immersed in a vessel containing water, and graduated so that the amount of displacement caused by this immersion could be measured. The weight of this displaced volume gave the loss of weight in water.

The specific gravity was obtained by dividing the dry weight in air by the displaced weight.



SPECIFIC GRAVITY APPARATUS

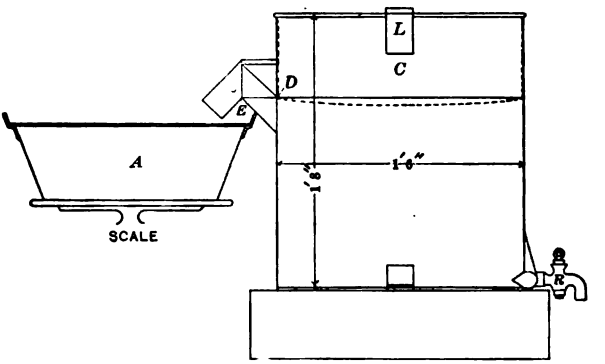


FIG. 20.

The determination for the large cubes was made in a special apparatus, shown in Fig. 20. This apparatus consists of a heavy sheet-copper tank, tinned on the inside, 1 ft. 6 ins. in diameter and 1 ft. 8 ins. high. Into the top of this tank fits a cup, 6 ins. deep and ¼ in. less in diameter than the tank. Across the top of this cup is fastened a brass strip, which projects beyond the sides of the cup. When the latter is in position the brass strip rests on two lugs on the sides of the tank. The bottom of the cup extends ⅛ in. below the normal

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level of the water in the tank. The cup is made convex on the bottom to prevent the collection of air bubbles. The normal level is maintained at the same height by means of an L-shaped spout having a 2-in. circular orifice. The lower point of the elbow of the spout is ¼ in. below the upper edge of the opening made by the spout into the tank. A spigot at the lower part of the tank permits of emptying the tank at the end of the tests. Mr. Humphrey.

To use the apparatus, the cup is lifted out and the tank is filled with water until it overflows through the spout. The cup is then placed in position, and brought to a bearing on the brass lugs. The cup forces out a small quantity of water, and changes the level of the water in the tank from a variable level to a constant artificial level.

An aluminum pan, having a capacity of about 3½ galls., is placed under the spout. The cup is carefully lifted from the tank and placed over the pan, so that the drippings are not lost.

The test specimen, treated as previously described, is then placed in the tank, and the cup is gently replaced, thus restoring the previous artificial level. The water in the pan is that which has been displaced by the test piece. The dried weight in air divided by the weight of the displaced water in the pan gives the specific gravity.

Weight per Cubic Foot.—This was obtained from the volume of the water displaced (in the specific gravity determination), reduced to inches, and the dried weight in air.

Crushing Tests.—The smaller cubes were tested on the 300 000-lb. Olsen machine in the Philadelphia Testing Laboratory; the large cubes were tested on the 2 160 000-lb. hydraulic machine of the Phœnix Iron Works, Phœnixville, Pa. This machine consists of a cylinder, in which works a piston with four piston rods. At one end of these rods is attached the piston head, and to the other the cast-iron head for holding the test pieces for tension tests. The other head is adjustable, but remains fixed during the tension tests. For making the crushing tests, the cubes were placed between the cast-iron end of the cylinder and back of the cast-iron head attached to the four piston rods.

The cylinder is filled with water by means of a steam-driven pump, and, by an arrangement of valves, the piston head can be moved in either direction. The pressure is recorded by means of a mercury column.

An experimental set of cubes was first crushed, using the plane cast-iron faces of the testing machine, and with no cushioning on the test specimen. This proved unsatisfactory, because the bearing faces of the cubes, although plane, were not parallel, and an imperfect bearing resulted, which caused the specimens to crush on the side having the greatest depth.

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The use of plaster of Paris to form a more perfect bed was found to be a slow process, and the results were not very satisfactory.

Again, ¼-in. white pine boards were tried on both crushing faces, and proved even less satisfactory.

Finally, a cast-iron head having a ball and socket bearing was attached to the back of the movable head. This gave an excellent bearing, and no bedding was required for the test specimen. The bearing faces of the cubes, however, were rubbed to as nearly perfect planes as possible. The method for the 2-in. cubes was the same, except that a double layer of blotting paper was used to bed the crushing faces.

The results of the tests of the 8-in. cubes are shown in Table No. 9, and those of the 2-in. cubes in Table No. 17. They represent a fair range of the qualities of building stones, from sandstone to granite. The character of the breakage of the various cubes is shown in Plates XXV and XXVI.

Generally, the breakage was of one of the following classes:

1. In the form of a single pyramid, usually found on the moving head of the testing machine;

2. In the form of a double pyramid; (*a*), one pyramid sliding over the other, or (*b*), the pyramid formed on the moving head forming a crater in the opposite pyramid;

3. Specimen failing all over, or splitting up in lines parallel to the line of pressure.

The sandstones yielded breakages of the first and second classes, while the laminated stones, tested on edge, gave breakages of the third class.

Figs. 1 to 5, Plate XXV, show the result of the tests of the Curwensville stone; a very homogeneous, light-colored, fine-grained, compact sandstone. The specimen broke in a pyramidal form in each case.

Figs. 1 to 3 were tested on natural bed; Figs. 4 and 5 were tested on edge.

Fig. 3 shows a breakage of the second class, the pyramid formed on the breaking head having formed a crater in the opposite pyramid.

Unfortunately, the 8-in. specimens, through oversight, were destroyed before they could be photographed. Their breakage, however, was quite similar to that of the 2-in. cubes.

Figs. 6 to 10, on Plate XXV, and 14 to 17, on Plate XXVI, were of French Creek granite, a thoroughly homogeneous syenite (without grain or apparent laminations), fine, close-grained and dark-colored.

The stone showed a tendency to fail in lines parallel to the line of pressure, although the pyramidal breakage is apparent in Figs. 10 and 17.

Each cube failed with a loud report, and was accompanied by a splintering, and not by a pulverization, of the small pieces, as was the case with the cubes of Curwensville stone.

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TABLE No. 17.—RECORD OF TESTS OF STONE USED IN THE CONSTRUCTION OF THE PENNSYLVANIA AVENUE SUBWAY.

TESTS MADE ON TWO-INCH CUBES.

Mark.	Name of stone and location of quarry.	Dimensions of specimen.	Weight per cubic foot.	Absorption, per-centage of.	Specific gravity.	Compressive Strength, in Pounds.		How Tested.		Average Compressive Strength, in Pounds per Square Inch.		Remarks.
						Ultimate.	Per square inch.	Bed or edge.	Grain.	On bed.	On edge.	
A1....	French Creek.	1 31/32 x 1 31/32 x 2	190.65	0.17	3.07	76 870	19 838	bed.	none.	19 997	14 348	Plate XXV, No. 6
A2....		1 31/32 x 1 31/32 x 2	198.66	0.16	3.11	77 490	19 990	"	"			" " " 7
A3....		2 x 2 x 2 1/32	192.91	0.15	3.10	52 080	13 020	edge.	"			" " " 8
A4....		2 x 2 x 2 1/32	192.61	0.13	3.09	80 660	20 165	bed.	"			" " " 9
A5....		1 31/32 x 1 31/32 x 2 1/32	190.58	0.14	3.08	59 790	15 675	edge.	"			" " " 10
C1....	Rittenhouse.	1 7/8 x 1 7/8 x 1 31/32	178.80	0.14	2.88	52 540	14 459	bed.	with.	19 891	15 555	Plate XXV, No. 11
C2....		1 7/8 x 1 7/8 x 1 7/8	176.05	0.14	2.80	75 350	21 797	"	"			" " " 12
C3....		1 7/8 x 1 7/8 x 2	176.13	0.11	2.82	82 330	23 416	"	"			" " " 13
C4....		1 31/32 x 1 29/32 x 1 15/16	176.69	0.13	2.81	43 520	11 790	edge.	against.			" " " 14
C5....		1 7/8 x 1 7/8 x 1 31/32	175.91	0.13	2.80	67 910	19 390	"	"			" " " 15
D1....	Curwensville.	1 31/32 x 1 31/32 x 2	146.55	2.25	2.40	30 980	7 793	bed.	with.	10 218	8 018	Plate XXV, No. 1
D2....		2 x 1 31/32 x 1 31/32	148.14	2.36	2.39	38 490	9 775	"	"			" " " 2
D3....		1 31/32 x 2 x 1 31/32	148.18	2.45	2.37	51 590	13 085	"	"			" " " 3
D4....		1 31/32 x 1 31/32 x 1 31/32	147.21	2.44	2.38	31 880	8 295	edge.	against.			" " " 4
D5....		2 x 1 31/32 x 2	147.67	2.36	2.37	30 290	7 800	"	"			" " " 5
E1....	Leiperville.	2 1/32 x 1 31/32 x 2	167.31	0.39	2.70	21 960	5 657	bed.	"	6 097	5 446	Plate XXV, No. 16
E2....		2 x 2 1/32 x 1 31/32	167.88	0.37	2.71	27 464	6 760	"	"			" " " 17
E3....		2 x 1 31/32 x 1 31/32	165.41	0.35	2.68	22 760	5 874	"	"			" " " 18
E4....		1 31/32 x 1 31/32 x 2	167.05	0.37	2.69	22 174	5 813	edge.	with.			" " " 19
E5....		1 31/32 x 2 1/32 x 1 31/32	165.13	0.36	2.68	20 310	5 079	"	"			" " " 20
F1....	Conshohocken.	1 31/32 x 1 31/32 x 1 31/32	178.70	0.15	2.91	82 080	22 598	bed.	with	20 088	15 680	Plate XXV, No. 21
F2....		1 31/32 x 1 31/32 x 2	178.75	0.15	2.88	62 120	16 285	"	"			" " " 22
F3....		2 x 2 x 1 31/32	177.47	0.16	2.90	84 940	21 235	"	"			" " " 23
F4....		2 1/32 x 2 1/32 x 1 31/32	178.90	0.15	2.93	85 050	20 301	edge.	against.			" " " 24
F5....		1 31/32 x 2 1/32 x 1 31/32	177.42	0.14	2.89	42 180	11 058	"	"			" " " 25

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Mr. Humphrey.

The tests of Rittenhouse stone, a dark-colored, fine-grained gneiss, are shown in Figs. 11 to 15, Plate XXV, and Figs. 10 to 13, Plate XXVI.

Cubes 12, 14 and 15 gave breakages of the third class, while Cubes 10 and 13 (Plate XXV) gave breakages of the first class; the breakage of Cube 13 was almost a perfect pyramid, having been formed on the moving head.

Leiperville stone is shown in Figs. 16 to 20, Plate XXV, and Figs. 6 to 9, Plate XXVI. This is a light-colored gneiss, and varies considerably in hardness, the 2-in. being not as hard as the 8-in. specimens. The breakages of Cubes 16 to 18 show distinct vertical splitting in the line of pressure. Cube 8 is an exception to the third class, the breakage being in the form of a pyramid, although the test was made against the grain. This is reported as having been made on bed, as the proprietors of the quarry claim that the natural bed is against the grain. Cube 6 was also on this bed, the wedge-shaped piece having been formed on the moving head.

The breakages of the Conshohocken stone, a very hard mica schist of a medium dark color, are shown in Figs. 21 to 25, Plate XXV, and Figs. 1 to 5, Plate XXVI. These cubes also failed with a loud report.

The smaller cubes (Figs. 21 to 25) broke rather irregularly, Cube 21 making the best showing, the breakage being in the form of a fairly good pyramid. Cubes 1 and 2 show failure by splitting in the line of pressure, the former breaking in the shape of a rather narrow pyramid, formed on the moving head. The breakages of Cubes 3 and 5 were pyramidal in form.

The average results of these tests, of both 2-in. and 8-in. specimens, are shown in Table No. 18.

TABLE No. 18.—Comparison of the Averages of the Tests on Eight and Two-Inch Specimens of Stone.

Name of stone.	Approximate dimensions.	Weight, per cubic foot.	Absorption, percentage of.	Specific gravity.	Compressive Strength, in Pounds per Square Inch.	
					On bed.	On edge.
French Creek...	8-inch.	188.84	0.16	3.08	17 274	7 910
	2-inch.	192.08	0.15	3.09	19 997	14 348
Rittenhouse.....	8-inch.	175.84	0.14	2.84	11 686	13 984
	2-inch.	176.61	0.13	2.81	19 891	15 555
Curwensville.....	8-inch.	144.44	2.30	2.42	7 513	4 462
	2-inch.	147.55	2.37	2.38	10 218	8 013
Leiperville	8-inch.	164.96	0.20	2.69	9 505	6 496
	2-inch.	166.45	0.57	2.69	6 097	5 446
Conshohocken...	8-inch.	177.27	0.16	2.92	10 417	7 532
	2-inch.	178.25	0.15	2.90	20 036	15 080

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PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLVIII, No. 930.
HUMPHREY ON TESTS OF STONE.

CURWENSVILLE.

FRENCH CREEK.

RITTENHOUSE.

LEIPERVILLE.

CONSHOHOCKEN.

RESULTS OF CRUSHING TESTS ON TWO-INCH CUBES.

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Mr. Humphrey.

In closing these remarks, it might seem proper to draw some conclusions from the results of these tests; such is not the intention, however, as it is believed that the number of tests is entirely too small to permit of any general conclusions being drawn therefrom. Discussion of these results, however, seems quite admissible.

In a general way, it would seem from these average tests that the compressive strength per square inch of surface decreases as the size of the cube increases, and that those stones yield the greatest compressive strength per square inch which have the least absorption, the highest specific gravity, and the greatest weight per cubic foot.

General Gillmore, from the results of his tests (reported in 1875) on cubes of Berea sandstone, varying in size from ½ in. to 4 ins., reaches the conclusion that "the compressive resistance of cubes per square inch of surface under pressure increases in the ratio of the cube roots of the sides of the respective cubes, expressed in inches." He also found that the theoretical curve was that of a cubic parabola. From which he concludes:

"That if certain cubes of unit dimensions are built together, with cement equal to their own substance, into a cube of large dimensions and of homogeneous strength, the resistance to compression per square inch of bed-surface increases as the half-ordinates of a cubic parabola."

His later experiments, in the same year, on 11-in. cubes of the same stone, failed to confirm this theory.

In his book* he states that his later experiments failed to show a continuance of this law beyond the small cubes.

By way of explanation of this failure, General Gillmore states as follows:

"In preparing small cubes for tests, the soundest pieces are necessarily selected; any material in which flaws, hair cracks, or any other deficiencies can be detected on careful examination, is rejected. The test-piece is naturally designed to be a perfectly sound specimen of its class. Within rather narrow limits, it is possible that, owing to such careful selection, pieces of the same kind of material but of varying sizes are uniform as to texture and identical in homogeneity, and under such conditions it may be taken for granted that some law approximately applies.

"The difficulty of close examination and proper selection increases with the greater size of cubes. The stone appears, perhaps, on the outside, quite sound and of uniform texture, but through its mass it may want homogeneity of structure; the material cementing together the grains may be weak in parts, and the grains themselves of varying strength; and there may be cavities, cracks, and soft patches inside of the mass. These defects can be discovered when a large block is split to cut it into smaller cubes, for which the soundest parts are chosen; but the probability that the specimen contains unsound parts

* "Notes on the Compressive Resistance of Freestone, Brick Piers, Hydraulic Cements, Mortars and Concretes," 1890.

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Mr. Humphrey.

increases with the size. This will also explain the fact that cubes of the same size and kind occasionally vary greatly in strength. * * *

"In practice, a comparatively large cube ceases to be a unit, but is rather a conglomerate of smaller irregular pieces, joined together by a cementing substance of varied strength, and perhaps partially separated by minute cracks, cavities or pores. Under such conditions the stone cannot develop the same strength as if it were a true unit."

The results of the tests of the stone used in the subway fail to confirm General Gillmore's theories in several particulars.

In the first place, the larger cubes gave uniformly lower tests than the smaller cubes; the exception being in the case of Leiperville stone, which was probably due to the fact that the stone from which the smaller cubes were made was inferior to that from which the larger cubes were made.

In the second place, it does not seem logical to ascribe the failure of a cube to develop a greater strength in the larger cubes to lack of homogeneity and flaws, when such stones as French Creek, Rittenhouse and Curwensville, which are perfectly homogeneous and uniform as to grain and soundness, show a lower strength in the larger cubes.

It is very difficult to state the causes of the irregular tests of cubes of stone. It is evident, however, that it is folly to attempt to draw deductions from such tests, unless they be very considerable in number and the individual results of the tests agree well among themselves.

Again, it does not seem proper to select the finest quality of stone obtainable from which to prepare the test piece.

The stone is generally used in large pieces, and as it comes from the quarry, and it would seem logical, therefore, in selecting the samples from which tests are to be made, to take pieces representative of the poorest as well as of the best qualities of the material. Inasmuch as the object of such tests is to obtain a fair average value of the quality of the stone, a maximum and a minimum value should be reported. And, for the reason that stone is used in large masses, it would also seem logical to use for the test piece as large a cube as the facilities at hand will permit.

The engineer is interested in knowing, not what test carefully selected specimens of the highest quality will stand, but rather what can be expected of the stone as it comes from the quarry, and when used in large masses.

The preparation of the test cube is exceedingly important, and should be made with extreme care, inasmuch as irregularities in the crushing faces vitally affect the results of the tests. Yet it is almost impossible to secure perfect test specimens. Theoretically, the cubes should be sawed out to nearly the desired size, and then rubbed down to the proper dimensions on a rubbing table.

Some of the stones tested (French Creek, Rittenhouse, etc.), were

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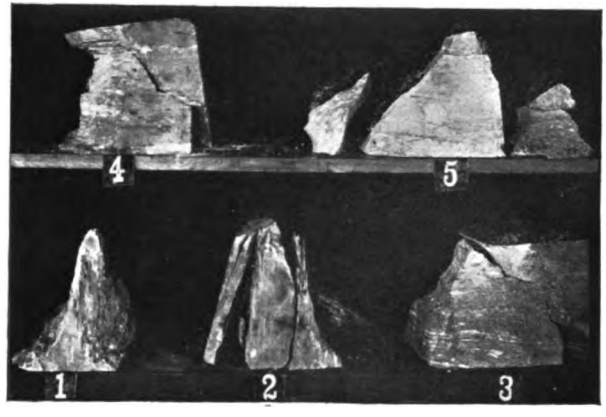
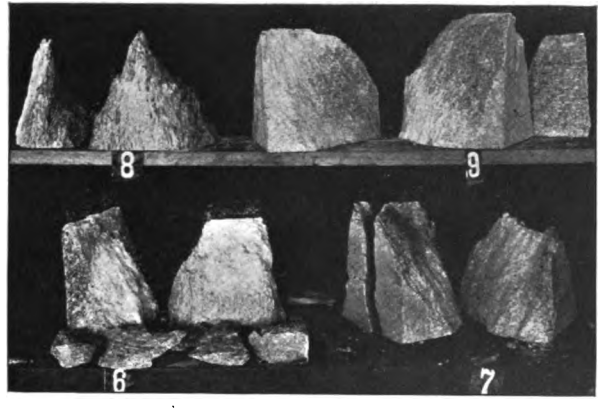


CONSHOHOCKEN.



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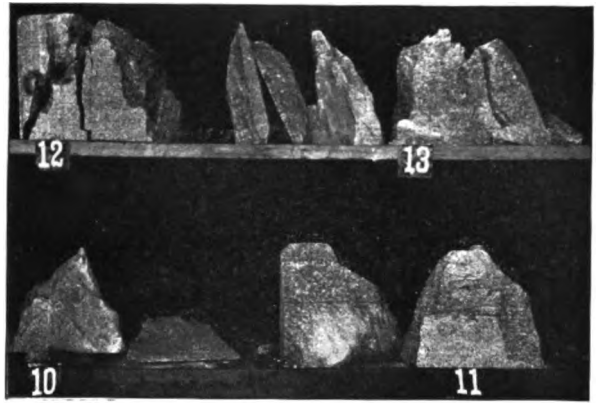
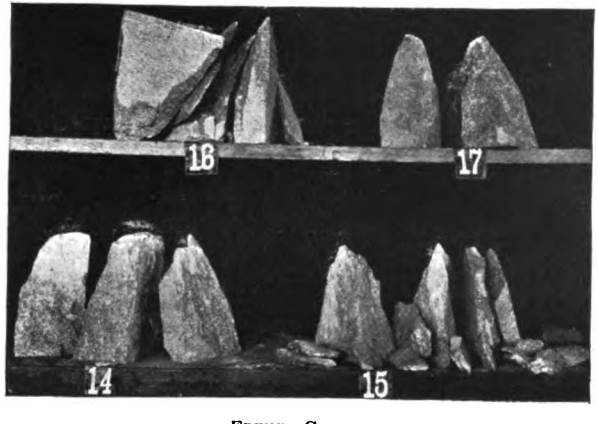


RITTENHOUSE.



FRENCH CREEK.

RESULTS OF CRUSHING TESTS ON EIGHT-INCH CUBES.

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so hard that the ordinary stone-saw made practically no impression after several hours' work. It was found impracticable, therefore, to saw these cubes out, as no stoneyard was willing to bother with the work; besides, the cost would have been quite high. Mr. Humphrey.

Although some of the softer stones could have been sawed out, it was decided, for the sake of uniformity, to have a skilled stone-cutter get the cubes out to approximate dimensions with a hammer and chisel, and finish them on the rubbing table.

The effect of the stone-cutter's hammer on the test specimen is problematical. It is probably true, however, that the structure of the stone is injured by the shock produced by repeated blows of the hammer, the adhesion of the particles of which the stone is composed is disturbed, to a greater or less degree, and the strength of the stone is therefore impaired.

The amount of this damage is difficult to determine. It would seem fair to presume that the effect of hammering would be greater on the 2-in. than on the 8-in. cubes; but the results of the tests do not seem to bear out this theory. However, the test specimen should be sawed to approximate dimensions, whenever practicable, and finished by rubbing to plane surfaces of the desired size.

The whole question of the methods of testing the qualities of building stone is an open one, and there seems to be very little information available on the subject.

It seems strange that in past years scientific knowledge and skill have been concentrated on the development of the properties of iron and steel, while comparatively little attention has been paid to the properties of building stone.

Some of the reasons for this failure to carry on investigations of this character may be stated briefly, as follows:

1. Difficulty in securing test specimens;
2. Lack of facilities and apparatus of sufficient capacity for the work; and,
3. Lack of time.

An exhaustive series of tests on the properties of the building stones of the United States would furnish exceedingly valuable data, and would meet a long-felt want.

Such a series of tests could be very properly carried on in a Government laboratory. There is a constantly growing need for such a laboratory, where the investigation of the properties of all the materials of construction could be conducted. It is to be hoped, therefore, that this need will crystallize in such definite shape that it will lead to the establishment of a laboratory by the Government in the near future.

GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER, Members, Am. Soc. C. E. (by letter).—A closer examination of the records of the first designs for the tunnel and its ventilation show that, as first planned, Messrs. Webster and Wagner.

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no provision whatever was made for either an artificial system or for openings in the street above. The reason for so doing was the fact that the tunnel of the Baltimore and Ohio Railroad, which parallels the Subway Tunnel for a considerable distance, and which is about 600 ft. longer,* has never been the subject of any complaint, in spite of the fact that in its alignment there is an 8° curve, 698 ft. long, located at about the center between two tangents, and that the prevailing winds are in the direction of the Subway Tunnel for eight months in the year. Anemometer readings made in the Baltimore and Ohio Tunnel, showed that in nearly all cases a velocity of 2 ft. per second was obtained, even when the direction of the wind was at right angles to the tunnel. Based on these experiments, which were made by the Bureau of Surveys, it was shown that, with the wind on the surface blowing at the rate of 9.8 miles per hour, and using the lowest ratio obtained between the surface and the underground velocities, the air in the Subway Tunnel would be naturally changed in 10 minutes. With a velocity of but one-sixth of this amount it would be changed in 20 minutes.

At a conference held in August, 1894, between the officials of the City and the Railroad Company, the latter asked that a series of openings be left along the center of the street over the tunnel, and that they be protected with iron fences and filled in with shrubbery so as to mask and conceal the openings, and, further, that there be at least eight such openings along the line of the tunnel.

This proposition was objected to at the time by the officials of the City, the result being the formulation of the artificial system which has been described at length and which in turn was abandoned for practically what the Railroad Company asked in the first place. It is of interest to note that during the past three years, in which the tunnel has been in continuous operation, no complaint whatever has been made by the Railway Company on account of insufficient ventilation. In fact, a recent discussion of the matter with the officials of the Company resulted in the statement that the ventilation had been entirely satisfactory, from an operative standpoint, and that they had obtained the system that they originally desired.

Recent personal observation also reveals the fact that the air in the tunnel is wonderfully free from objectionable features, and to the passenger on a train during the time of the year that the windows are open there is no discomfort whatever. The light from the openings is also sufficient to take away the unpleasant feature of the extreme darkness generally found in passing through a tunnel.

During the discussion of the tunnel designs many data were obtained as to the general question of ventilation, some of which are not out of place in the present discussion.

* *Transactions*, Am. Soc. C. E., Vol. xxvi, page 530.

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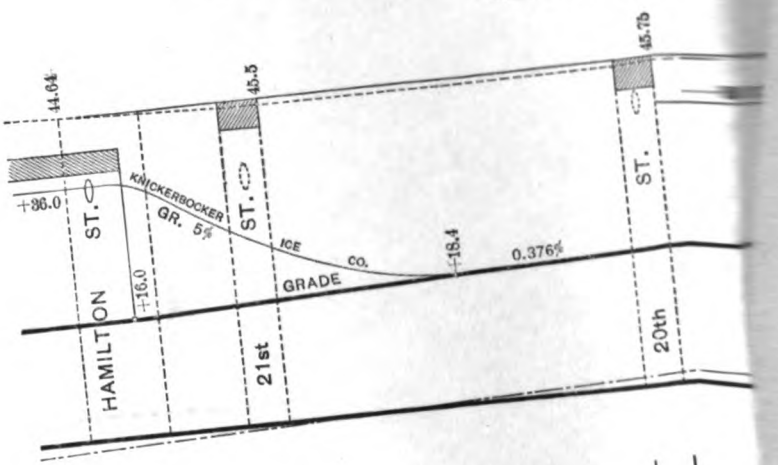

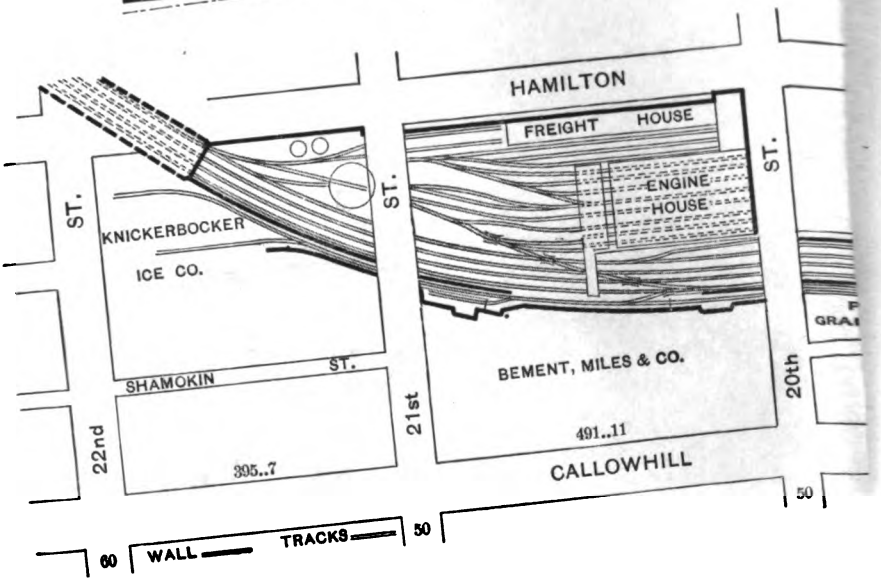

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The following is an abstract from a paper* by Francis Fox, M. Inst. C. E.

"The Author has by inquiry and by personal experience ascertained that if the amount of carbon dioxide can be kept low, so as not to exceed 20 parts in 10,000, the air in a tunnel is satisfactory.

"The consumption of coal by a locomotive during the passage through a tunnel having been ascertained, allowing 29 cubic feet of poisonous gas for each pound of coal consumed, the volume of fresh air required to maintain the atmosphere of the tunnel at the above-mentioned standard of purity is ascertained as follows: The number of lbs. of fuel consumed per mile, multiplied by 29, multiplied by 500, and divided by the number of minutes' interval between the trains, will give the volume of air in cubic feet which must be introduced into the tunnel per minute to maintain it in a sufficiently pure state to avoid inconvenience. * * *

"This is the basis upon which the Mersey and the Severn Tunnels are ventilated, and which have been shown, by the report of the Committee referred to, to be efficiently supplied with fresh air; it is, however, to be regretted that, in the case of the Mersey Tunnel, owing to the impecuniosity of the Company, inferior coal is now used, and the fans are driven at a lower speed than was intended. The airways then become encrusted with soot, which in some places is between 2 inches and 3 inches in depth; but, notwithstanding these drawbacks, the examination of the air of the tunnel, made by Dr. John Haldane, F. R. S., for the Departmental Committee, showed that the amount of carbon dioxide was considerably less than 20 parts in 10,000. There is no doubt that this tunnel, when proper coal is used and the fans are run at the intended speed, is the most perfectly ventilated tunnel of its kind in the world. * * *

"The Mont Cenis Tunnel is about 8¼ miles in length and 26 feet in width, and, in consequence of the mountain above it, possesses no shaft. * * * At the Bardonecchia entrance there is a somewhat obsolete installation of seven overshot water-wheels, 5 metres in diameter, and 6 metres in length on the breast, over which in succession a mountain stream is conducted. These wheels work air-compressors, the air being delivered under a pressure between 4½ and 6 atmospheres, and discharged at points 1 kilometre apart throughout the tunnel. The quantity of air is so small that for purposes of tunnel ventilation it has little effect, amounting only to 70 cubic metres per minute of air at 75 lbs. pressure per square inch; but it is of great advantage to the men employed in the tunnel. * * *

"The St. Gothard Tunnel is 9⅓ miles in length, and 26 feet in width; it is nearly level from end to end, the gradient being only that needed for drainage purposes. The ventilation was entirely natural until recently; but, owing to the use of briquette fuel, with its dense smoke, and to the constant increase of the traffic, a time arrived when it was necessary that something should be done, and recently the Saccardo system has been adopted.

"In long tunnels on steep gradients, such as exist in Italy and elsewhere, artificial ventilation is imperatively needed. In the cases of the Pracchia Tunnel, 3,000 yards in length on a gradient of 1 in 40; the Mont Cenis Tunnel, with its peculiar longitudinal section, * * * with gradients of 1 in 40 at both ends; the spiral tunnels, with grad-

*"Ventilation of Tunnels and Buildings," *Minutes of Proceedings*, Inst. C. E., Vol. cxxxvi, Session 1898-99, Part II.

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ients of 1 in 40, on the St. Gothard route, and the long tunnel of Ponte Decimo near Genoa—grave inconvenience and even worse results arise. Asphyxiation is not unknown, and the loss of life attending the collision of the 11th August, 1898, at the mouth of the last-named tunnel, in which twelve persons were killed and forty injured, was the direct result of bad air.

"The case of the Pracchia Tunnel is interesting. It is one of fifty-two tunnels on the main line between Florence and Bologna, built by the late Mr. Thomas Brassey, Assoc. Inst. C. E. These are single-line tunnels, on a gradient of 1 in 40. The traffic has increased greatly and has to be worked by heavy locomotives. Under any condition of wind the state of this tunnel, about 3,000 yards in length, is bad; but, when the wind is blowing in at the lower end at the same time as a heavy goods or passenger train is ascending the gradient, an almost insupportable state of affairs is produced. The engines, which are working with the regulators full open, often emit a large quantity both of smoke and steam, which travels concurrently with the train. The locomotives weigh 55 tons without tender. They have eight wheels coupled, with a tractive force of 15,400 lbs. The goods trains carry 250 tons of load and have an engine in front and one at the rear of the train; and when, from the humidity in the tunnels due to the steam, the wheels slip and possibly the train stops, the condition of the air is indescribable. A heavy train with two engines, conveying a Royal party and their suite, arrived on one occasion at the upper exit of the Pracchia Tunnel with both engine-men and both firemen insensible; and on another occasion, when a heavy passenger train came to a stop in the tunnel, all the occupants were seriously affected. On the occasion of the Author's visit to the tunnel, on the 25th July, 1894, one of the brakesmen complained of being ill and of having fallen from his seat in one of the shorter tunnels.

"The Saccardo system of ventilation has just been applied to the Pracchia Tunnel with remarkable results. Mr. Marco Saccardo, the well-known Italian engineer, avails himself of the annular space, between the interior surface of the tunnel and the gauge of maximum construction, and, upon the principle of the injector, blows, by means of a fan, a large volume of air into the tunnel at its mouth. This induces a strong inward current at the central opening, through which the trains pass. The Author measured the volume and temperature of air with the following result. Before starting the fan, the tunnel was filled with dense smoke from end to end, the temperature being 107° F., with 97 degrees of moisture, or nearly complete saturation. With the fan running, the thermometer indicated 80° (the temperature of the external air), or a fall of 27°, the moisture was normal, and the amount of air propelled by the fan was 164,000 cubic feet per minute, that by the induced current being 46,000 cubic feet, making a total of 210,000 cubic feet of air per minute passing through the tunnel. The air is blown in at the upper end, and down the incline, the object being that an ascending train with its heavy trail of smoke and steam may be freed at the earliest possible moment from the products of combustion. These results are remarkable, and the air of the tunnel is cool and fresh. This system, however, cannot be applied to railways in which underground stations exist, as the effect of the current would be to blow the smoke to the platforms of the next station, the very part of the railway which should be kept in the best condition."

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Mr. Fox also writes the following on ventilation, in his paper entitled "The Mersey Railway:"*

"The only practical method of dealing with the impure air in such cases is, in the Author's opinion, the adoption of ventilating fans placed about midway between the stations, by which a steady and continuous current of fresh air will flow in at each station, and thence through the tunnel to the fan. The air throughout the tunnel is thus changed, and not merely churned backwards and forwards."

William Barclay Parsons, M. Am. Soc. C. E., states as follows,† in reference to the ventilation of the Chemin de Fer de Sceaux in Paris:

"The Place Denfert and Port Royal Stations being open, it has been considered that frequent blow-holes will sufficiently ventilate the stretch of tunnel 1 900 ft. long between them. For this purpose holes have been left in the roof of the tunnel at intervals of from about 200 to 300 ft. Where these holes pierce the sidewalk ornamental glass and iron kiosks, * * * 8 ft. by 5 ft. in plan, are built. The upper part is an iron grating through which the foul air will escape. According to Parisian custom these kiosks will be covered with advertisements. * * * The Luxembourg Station being entirely underground it is necessary to adopt more effective methods than simply blow-holes to ventilate it and the approaching tunnels. To accomplish this there is established at this station a ventilating fan and an upcast chimney. On the haunch of the tunnel arch there has been built an air passage varying in section from 21½ sq. ft. to 86 sq. ft. for a distance of about 800 ft. from the station. This passage is connected at intervals with the tunnel through openings in the roof. On the other side there are fresh air inlets which come in at the rail or platform level and are connected at the street surface with kiosks, which in this case act with a down draught. The fresh air inlets and ventilator openings are never set opposite each other, but are 'staggered.' The action of the fan is therefore to remove the foul air from the top of the tunnel, the place of which is supplied by fresh air entering at the bottom."

The ventilating conduit and the air inlet are shown in Fig. 21.‡

In addition to the tunnels referred to by Mr. Soper, the following data were obtained, and are given in concise form as they existed in 1894:

EUROPEAN TUNNELS.

Box Tunnel.—Near Bath in England. Double track. Length, 9 600 ft. 92 trains per day. Two shafts, 25 ft. in diameter, used for ventilation. Would be better without the shafts.

Hauenstein Tunnel.—Swiss Central. Length, 8 189 ft. Three shafts sunk during construction, but afterward walled up. No record of any difficulty.

Simplon Tunnel.—Switzerland. Under construction. Length, 64 740 ft. Special and elaborate arrangements for ventilation during and after construction.

* *Minutes of Proceedings*, Inst. C. E., Vol. lxxxvi, p. 58.

† "Report to the Board of Rapid Transit Railroad Commissioners in and for the City of New York, on Rapid Transit in Foreign Cities," 1894, p. 48.

‡ Fig. 21 is a reproduction of Fig. 46 in Mr. Parsons' 1894 report.

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AMERICAN TUNNELS.

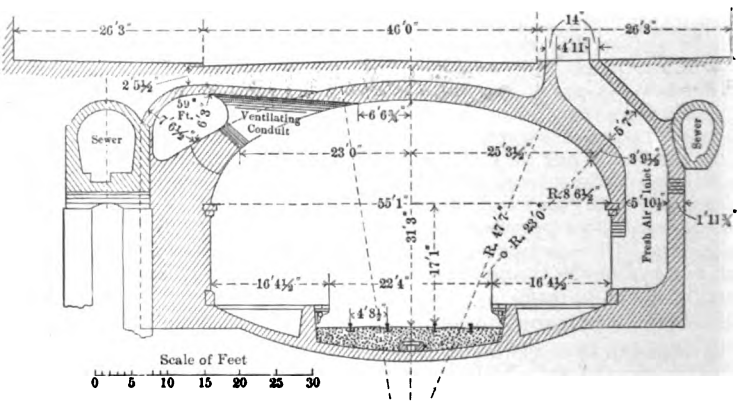
Bergen Tunnel.—Erie Railway. Double track. Length, 4 388 ft. 312 trains per day. Fans placed at two of the eight shafts. Ventilation somewhat improved. Fans discontinued on account of the expense. No serious trouble reported.

St. Louis Terminal Tunnel.—Double track, 4 095 ft. long. 167 trains per day. Fan and ventilating shaft introduced about the center, but inadequate for the purpose. Ventilation reported as very imperfect.

Weehawken Tunnel.—West Shore Railroad. Double track. Length, about 4 000 ft. Five shafts used in construction and afterward for operation. While open, ventilation poor. Shafts closed. Chief Engineer Walter Katté, M. Am. Soc. C. E., reports no difficulty in ventilation.

Baltimore and Ohio Belt Line.—Baltimore. Double track. Constructed with no provision for ventilation. Operated with electric engines.

Charles Street Tunnel.—Baltimore. Double track. Length, about 3 000 ft. Fan 10 ft. in diameter at central shaft, 65 ft. deep. Ventilation reported as unsatisfactory.



HALF SECTION THROUGH VENTILATING CONDUIT. HALF SECTION THROUGH AIR INLET.

CHEMIN DE FER DE SCEAUX. LUXEMBOURG STATION.

FIG. 21.

Mr. Parsons has called attention to the increasing use of concrete and steel in engineering construction, a matter which, at the present time, is compelling the attention of all constructors, on account of its great possibilities. At the time when the work under discussion was designed, the data at present available were not at hand, although the construction of the retaining walls of concrete was considered. It is doubtful, however, whether it would have been possible at that time to have obtained prices for the concrete in the walls which would have warranted the use of that material.

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Since that time, contractors and engineers have learned a great deal in reference to the best and most economical methods for handling this material, the result of which has tended to reduce the cost.

Under the laws of Pennsylvania, juries are appointed by the courts to hear testimony and to assess damages for the taking of land for any public use or for the change of grade of any street. When any land is appropriated, a jury of six is appointed by the Court of Quarter Sessions; for a change of grade a jury of three is appointed by the Court of Common Pleas. On the work on the subway both types of juries were at work, namely, for the widening of Pennsylvania Ave. west of Twenty-second St., and for the lowering of the avenue from that point to the east. An appeal from the award of the jury can be taken by either the City or the property owner, if so desired. All the legal work in connection with such cases falls upon the Department of the City Solicitor, and is attended to by his salaried staff. It is generally considered in the public mind that municipal work is extravagant, and for this reason attention is called to the items given in the statement on page 540. It will be noticed that the total expenses connected with the awarding of damages, amounting to $1 040 345.71, were but $33 011.05, which is a very small amount. The attention of the writers was called to this matter some time ago by the following statement:*

"Condemning Croton Lands. Expenses nearly equal the awards so far. Comptroller wants a better system.

"Comptroller Coler, who, with Corporation Counsel Whalen, holds that the present expensive method of condemning Croton Water-shed Lands should be done away with, prepared yesterday a statement of the cost of condemning these lands in the last seven years under the Webster Act of 1893. Mr. Coler's statement shows that the expenses of the Commissions of appraisal have been $918 798.48, and the awards for lands have amounted to $1 429 178.35. The expenses are thus tabulated:

"Costs and counsel fees allowed property owners..	$83 474.68
Commissioners' fees.....	373 820.32
Stenographers and typewriters.....	51 053.23
Clerks and messengers.....	61 777.99
Printing.....	63 927.71
Advertising.....	60 367.57
Experts.....	119 531.14
Rent.....	7 349.94
Miscellaneous expenses.....	16 145.16
Special counsel for City.....	112 394.20

"The Comptroller wants all the work done by a single Commission. He thinks that the Legislature ought to offer a remedy."

In dividing the work into contracts, it was thought best to separate, as far as possible, those contracts requiring special skill, such as the

* *The New York Sun*, October 24th, 1899.

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electric crane, locomotive coaling station, interlocking signals, etc., from the others which could be handled by a general contractor. The chief advantages which were to be derived from such an arrangement were the reduced cost, in the saving of an additional percentage, and being able to treat directly with the contractor. On the other hand, it was thought that probably there might be some difficulties caused by having two independent contractors working at the same time, with danger of their work overlapping. Even with the contracts covering the underpinning and retaining walls, it was thought by some that there might be danger of interference in having the general excavation of the core and the construction of the bridges under separate contracts. It so happened that, outside of one contractor having a number of the underpinning contracts, practically all the others were in the hands of different parties.

The results of the arrangements as laid out proved to be even more satisfactory than anyone connected with the work had anticipated. At times several contractors were working over the same territory, and the cases of dispute were surprisingly few. Most of the special work above referred to was contracted with specialists in the several lines, and the bidding almost invariably showed a saving in having the work divided so that the specialist could bid direct.

Great credit is due to the several contractors for the manner in which they worked together, and the willingness they always evinced in making preparations to begin work as soon as it was in any way possible to make a start on their contracts.

The inspection of the work was very thorough. The best inspectors of masonry and brickwork in the employ of the Bureau of Surveys were detailed on this work, and performed their duties faithfully under the direction of the engineers in the field. The inspection of the metal-work in the bridges, railings, buildings, etc., was placed in the hands of the most competent inspecting firms, with excellent results. Throughout the entire work of construction, Mr. E. S. Whitney, C. E., was assigned by the Philadelphia and Reading Railway Company to look after the details of construction in the field.

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